Fall 2009

Hunter College school of Social Work

Pro-Con Structural Study of Alternate Floor Systems

The Structural Concepts / Structural Existing Conditions Report consists of a Prepare a study and comparison of at least four different alternative floor framing systems for your building (one must be the original system). At least one must be a different framing material. In addition, no more than one system can be a variation on the same floor system.

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Fall 2009

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Technical Report 2 – Pro-Con Study of Alternate Floor Systems

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Executive Summary

In the second technical report alternative floor systems are investigated. A typical interior bay of 30'-0" x 38'-2" was analyzed and designed for four floor systems, including the existing, and were compared based on: self weight, total structural depth, constructability, impact on the existing architecture and steel structure, fire ratings, and cost. The existing floor system is composite and non-composite steel and was chosen because of its light self weight and ability to span long distances. The three other systems that are studied in this report include:

- Two-Way Flat Slab with Drop Panels
- Two-Way Post-Tensioned Slab
- Pre-Cast Hollow Core Planks on Steel Beams

The design of a two-way flat slab floor system resulted in a 10.5" thick slab, 13.5" thick to the bottom of the drop panels. This system was the most economical per square foot, however, the acquisition of a larger crane due to its 130psf self weight, and shutting down of 119th street would bring the cost way up. Therefore this is not considered a viable option.

For the post-tensioned two-way slab the design goal was to minimize the structural slab thickness. However, in order to support the loads a 10.5" thick slab was needed, increasing the self weight to 131psf; the largest self weight appearing for the alternative systems. Once again the large self weight of the system disqualifies it as a viable option for the floor system.

20'-0" long pre-cast hollow core planks were sized according to Nitterhouse Concrete Products Hollow Core Plank Design Tables and were determined to be 10" thick. 2" of lightweight topping was added to the hollow core planks to ensure a level floor surface. These planks are supported by W24x76 non-composite steel beams. The self weight of this system was found to 71psf using RSMeans 2009, compared to that of the existing system of 57psf it is considered a viable option since the designated crane size will suffice. Efficient manufacturing and construction methods, as well as long span capabilities, make pre-cast hollow core planks on steel beams a viable option for which a more in-depth economic and structural study is recommended.

Introduction

The building's design responds to the School of Social Work's mission by providing an open and engaging face to the neighborhood and opportunities for community use of parts of the facility. The entrance lobby, conceived as an interior street, is glazed from floor to ceiling along 119th Street to provide a transparent and welcoming appearance from the exterior and to link the interior of the building to its neighborhood surroundings. Classrooms and lecture halls occupy the lower levels with academic departments and offices on upper floors. An auditorium on the second floor is expressed on the facade, with a glazed wall allowing views of activity in and outside the building. A rear landscaped terrace will

Keyplan

The structure of Hunter College School of Social Work is comprised of a composite steel floor system that utilizes steel braced frames to resist lateral forces. Drilled caissons and spread footings make up the foundation system. The cellar floor is a reinforced slab on a mat foundation.

The purpose of Technical Report II is to examine alternative floor systems in efforts to discover a system that is a viable option for use within Hunter College School of Social Work in terms of cost, strength, and structural sandwich depth.

Code and Design Requirements

Applied to original Design

The Building Coded of the City of New York (most current) - Amended seismic design

AISC-LRFD, LRFD Specification for Structural Steel Buildings (applied except on the lateral force resisting frame)

AISC- ASD 1989, Specifications for Structural Steel Buildings- ASD and Plastic Design (for the design and construction of steel framing in lateral force resisting system)

ACI 318-89, Building Code Requirements for Structural Concrete

Substituted for thesis analysis

2006 International Building Code

ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Steel Construction Manual 13th edition, American Institute of Steel Construction

ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute

Material strength requirement summary

Structural Steel:

- All W Beams and Columns: ASTM A992, Fy=50ksi
- HSS Steel, Fy=46ksi
- Connection Material:Fy=36 ksi
- Base plates: ASTM 572 GR50, Fy=50ksi

Metal Decking:

- Units shall be 3" galvanized composite deck of 18 gage formed with integral locking lugs to provide a mechanical bond between concrete and deck

-Strength: Fy=40ksi

-Deflection of form due to dead load of concrete and deck does not exceed L/180 , but not more than ¾"

-Deflection of composite deck cannot exceed L/360 of deck span under superimposed live load.

Concrete:

-Caissons and Piers: 4000psi normal weight concrete

-Slabs on ground and footings: 4000psi normal weight concrete

-Retaining Walls: 4000 psi normal weight concrete

-Slab on deck: 3500psi lightweight concrete

- Foundations: 4000psi, air entrained, normal weight

-Walls, curbs, and parapets: 4000 psi

Reinforcement:

-Strength: 60ksi

Building Load Summary

Total building weight was found to be approximately 15,388kips. Detailed charts in Appendix A tabulate the columns and beams used in finding the total weight. Curtain wall weight was approximated to be 15 psf although curtain wall type varies as you go up in elevation. Glass curtain wall is used on the upper and lower sections of the building façade and precast masonry and stucco panels are used on the middle section of the building façade. Calculation of the building weight was tedious due to the varying bay sizes, column and beam sizes, and varying lengths of these members. In erection of the structure, careful coordination must be taken in order to correctly identify and place these frame elements.

Level	Floor Height (ft)	Slab Weight (lbs)	Column Weight (lbs)	Beam Weight (lbs)	Curtainwall Weight (lbs)	Total Level Weight (lbs)
Penthouse	134	80750	0	38245	0	118995
Roof	120	492300	3440	50726	70560	617026
8	104	403570	15938	37130	61740	518378
7	91	374170	24463	42135	57330	498098
6	78	1108370	24463	116396	127335	1376564
5	64	1201959	16940	169389	144690	1532978
4	50	1201959	86174	90008.7	144690	1522831.7
3	36	1201959	76816.5	140824.5	144690	1564290
2	19	3223770.5	76816.5	220889.5	178755	3700231.5
1	0	3356119.75	236557.1637	177844	168240	3938760.916
					Total Building Weight:	15388153.12

Figure 1. Building Dead Load Summary

ID	location	Live Loads (psf)			Dead Loads (psf)
		Design Live Loads	ASCE 705-05	NYC BLDG CODE 08	Design Dead Loads
1	loading dock	600	-	-	150
2	1st floor	100	100	100	130
3	podium	100	100	-	200
4	archive	350	-	-	75
5	offices	50	50	50	71
6	roof with garden	100	100	100	365
7	library stacks	100	100	100	71
8	classrooms	40	40	60	71
9	corridor	100	100	100	71
10	auditorium	60	60	100	85
11	roof with pavers on 2	100	-	-	150
12	roof	45	20	30	90
13	roof with drift	60	45	-	85
14	mechanical	100	125	100	120

Figure 2. Loading Schedule

Structural Systems

Foundation System

There is one below-grade level in the Hunter College School of Social Work. This level known as the cellar contains a parking garage for the residential building adjacent, a library, computer labs, large kitchen areas, and mechanical rooms.

Slab thickness varies throughout the cellar level. It can be 30", 33", or 40". Steel reinforcement varies according to the slab thickness. For a 30" slab #7@11 are required top and bottom (T&B) each way, for a 33" slab #8@13 top and bottom, and for a 40" slab #9@13 top and bottom each way. The mat foundation will have a 2" mud slab above 12" of ¾ crushed stone to facilitate installation of waterproofing membrane. The subgrade is composed of undisturbed soil or compacted back fill with a required bearing capacity of 1.5 tons.

The soil is not considered susceptible to liquefaction for a Magnitude 6 earthquake and a peak ground acceleration of 0.16g. It is expected to encounter ground water during erection of the cellar level. Excavation depths are anticipated to vary from about 12ft to 20ft below existing ground surface grades. Footings shall bear on sound rock with a bearing capacity of 20 ton per square foot or on decomposed rock with a bearing capacity of 8 ton per square foot or on sand with a bearing capacity of 3 ton per square foot.

Foundation walls are designed to resist lateral pressures resulting from static earth, groundwater, adjacent foundations, and sidewalk surcharge loads. These walls will extend 14ft below existing ground surface grades. Concrete for foundations and site work shall be air-entrained normal weight stone concrete with a minimum compressive strength of 4000psi at 28 days and a maximum water to cement ratio of 0.45 by weight.

In the western portion of the six story faculty housing building footprint, it is recommended to excavate rock 12" below bottom of foundation in order to limit differential settlement between sections of the mat foundation bearing on rock and that bearing on soil.

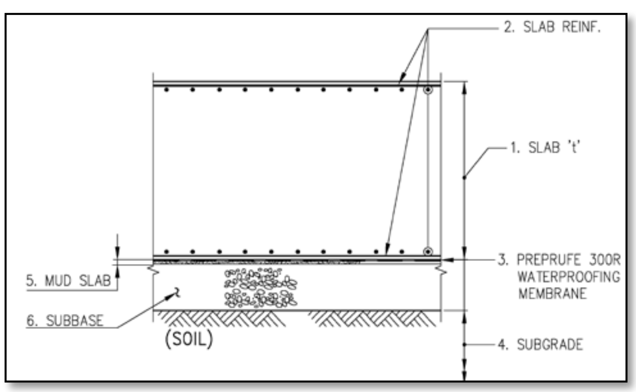


Figure 3: Mat Foudation Detail

Gravity System

Columns in the basement are 4000psi air-entrained concrete and vary in size from 32x48 to 36x60. The bay sizes vary from 30'x28', 30'x 28'2", 30'x31'5" and 30'x36' from north to south respectively.

All columns in the superstructure are W14s. Due to setbacks and varying story footprint, service loads carried by the columns at the ground level vary ranging from 137 to 1154kips. Because the service loads vary greatly throughout the floor, the column sizes vary as well; for example, on the ground floor column sizes range from w14x68 to w14x730. In the levels above the cellar, the bay sizes do not change.

There are non-composite beams as well as composite beams (with studs). Non-composite beams are found where beam to beam and beam to column connections are designed to transfer the reaction for a simply supported, uniformly loaded beam . For composite beams, connections are designed to have 160% capacity of the reaction for a simply supported, uniformly loaded beam of the same size, span, fy, and allowable unit stress. For framed beam connections, including single plate connections, the minimum number of horizontal bolt rows should be provided based on 3" center-to-center.

Roof System

The roof is typically composed of 3 1/2 " light weight concrete over 3"-18 gage metal deck reinforced with 6x6-2.9x2.9 WWF. In a 200 square foot section the slab is 8" lightweight concrete slab reinforced with #4@12 top and bottom E.W. Columns are placed where needed and don't necessarily follow a typical framing layout. To provide additional vibration control, 4" concrete pads are located below mechanical equipment. Curbs on the roof are of CMU and concrete.

Lateral System

Trusses with vertical members attached using moment connections make up the lateral system. Locations of these trusses are represented on figure 4 in red; they run all the way up the building levels. The only exception to this is — the frame truss represented on figure 4 as blue since it changes as you go up in elevation. An elevation view of this truss is shown as figure 5. Braced frames were chosen to resist lateral forces because they are more efficient than moment frames in both cost and erection time.

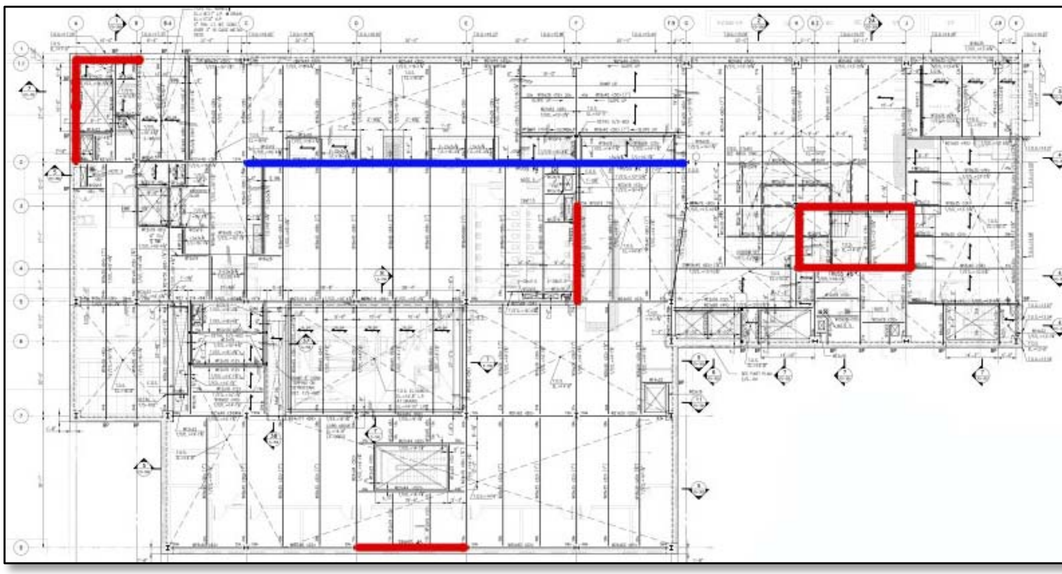
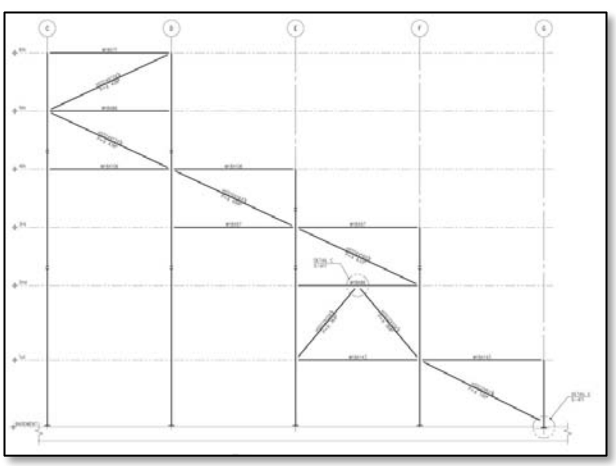


Figure 4. Location of Lateral Force Resisting Systems (Braced Frames)



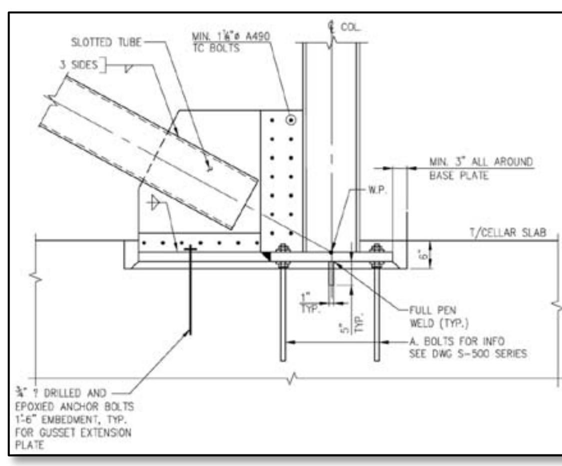


Figure 5. Truss Elevation at Grid 2

Figure 6. Lateral Load Resisting Detail

Floor Systems – System 1: Composite steel beam and deck floor system (existing)

The slab thickness for all floors is 3 ¼" thick 3500psi lightweight concrete placed over 3" deep 18 gage composite galvanized metal deck reinforced with 6x6- W2.9xW2.9 welded-wire-fabric. Exceptions on the ground floor are on the outdoor court, entry vestibules, and loading area; here 3" lightweight concrete is placed over 16 gage metal deck is used and instead of WWF, reinforcement is #4@12" o.c. top bars each way and 1-#5 bottom bars each rib. The exception for the second floor is the roof terrace where there is 5" of lightweight concrete over 3"-16 gage metal deck. On the roof level, the floor slab for the electrical control room is 8" lightweight concrete formed slab reinforced with to#4@12"o.c. top and bottom each way.

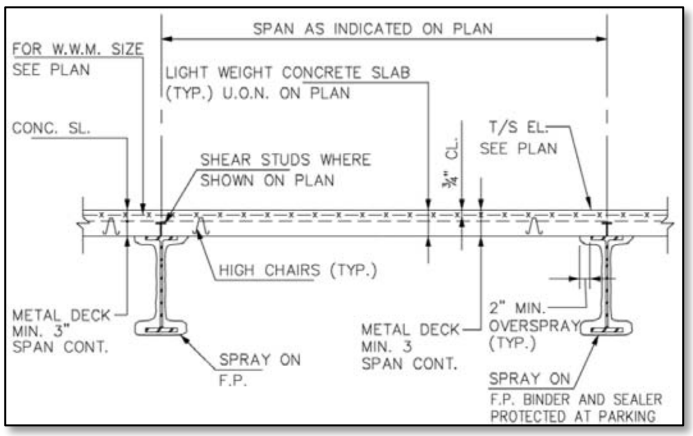


Figure 7. Typical Floor Construction , Metal Deck Perpendicular to Floor Beams on Girders

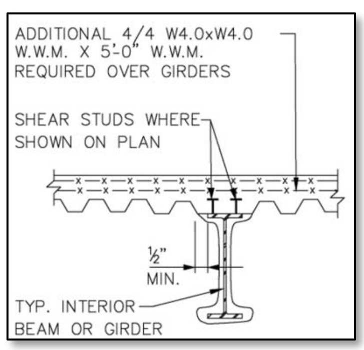


Figure 8. Typical Floor Construction, Metal Deck Parallel to Beams or Girders

Alternate Floor Systems

Alternative floor systems were analyzed for Hunter College School of Social Work. The main goal for the alternative systems was to reduce cost and structural sandwich thickness to increase floor to ceiling height. Bay sizes are kept the same due to the uniqueness of bay sizes throughout the building, bay sizes range from 28x30 to 38x30 to 31x30 , and to 15x30. These bay sizes vary due to the various community spaces and lecture halls.

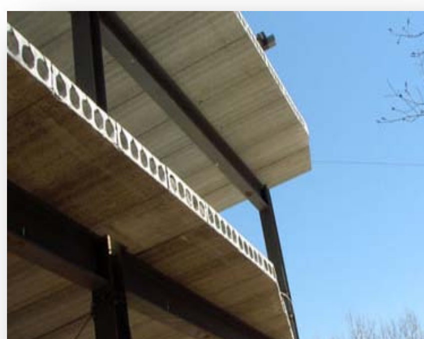
The systems that are analyzed within this report are (1) noncomposite and composites beam with metal decking, (2)two-way flat slab with drop panels, (3)two-way post tensioned concrete slab, (4) hollow core plank on steel beam. The systems discussed within this report were analyzed using the existing column grid. A typical interior bay used is 38'-2" by 30'-0". Various references were used in order to carry out the preliminary design of these systems

- AISC Specification for Structural Steel Buildings, 13th Edition
- ACI 318-08 Building Code and Commentary
- NitterHouse Hollow Core Plank Design Guide
- PCA Post Tensioned Slab Design Guide
- RS Means Assemblies Cost Data, 2009 Edition
- RS Means Building Construction Cost Data, 2009 Edition

System 1: (existing) beam with metal decking

System 2: flat slab with drop panel

System 3: Post Tension conc. slab



System 4: hollow core plank

System 2: 2way reinforced concrete slab with drop panels and flared column capitals

This system uses a two-way reinforced concrete slab to transfer gravity loads directly to columns. The presence of drop panels allows for a more slender slab since the area around the column has been strengthened to withstand the gravity loads and therefore the remaining part of the slab can be thinner since the load it sees is smaller than those near the column. A typical interior bay of 38'-2"x30'-0" was used to design the floor system since it was the largest bay size, therefore the most critical. To keep the slab thickness economical, it is assumed that all bays in the building will have the same slab thickness. A 2 hour fire rating was attained by providing a minimum clear cover of ¾" with carbonate aggregate.

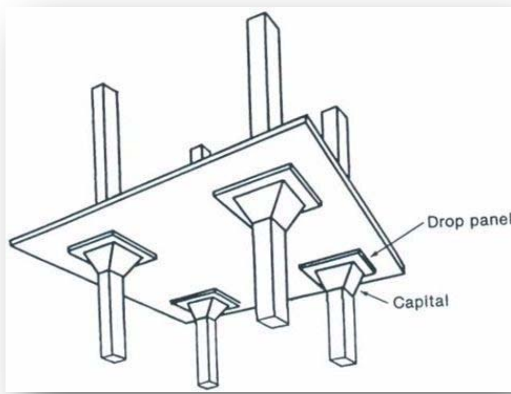


Figure 9. Two-way flat slab with drop panels (www.crsi.com)

Pro-Con Analysis: Two-Way Flat Slab Floor System

A two-way slab floor system works very well for the typical interior bay analyzed in this report. Even with drop panels added to prevent punching shear; the total structural depth is nearly half of the existing composite steel floor system. The flared columns however would impact the modern architecture of Hunter College School of Social Work.

Although this system is efficient for a typical interior bay of the of the building, an alternative to the current lateral load resisting steel frame would be needed. The additional weight of the concrete system would also change the foundation and cellar level structural frame.

Please refer to the nest two pages for the final design of the Two-way flat slab with drop panels

FINAL DESIGN: $t_{slab} = 10.5''$ DIAM. COL CAPS = 7'

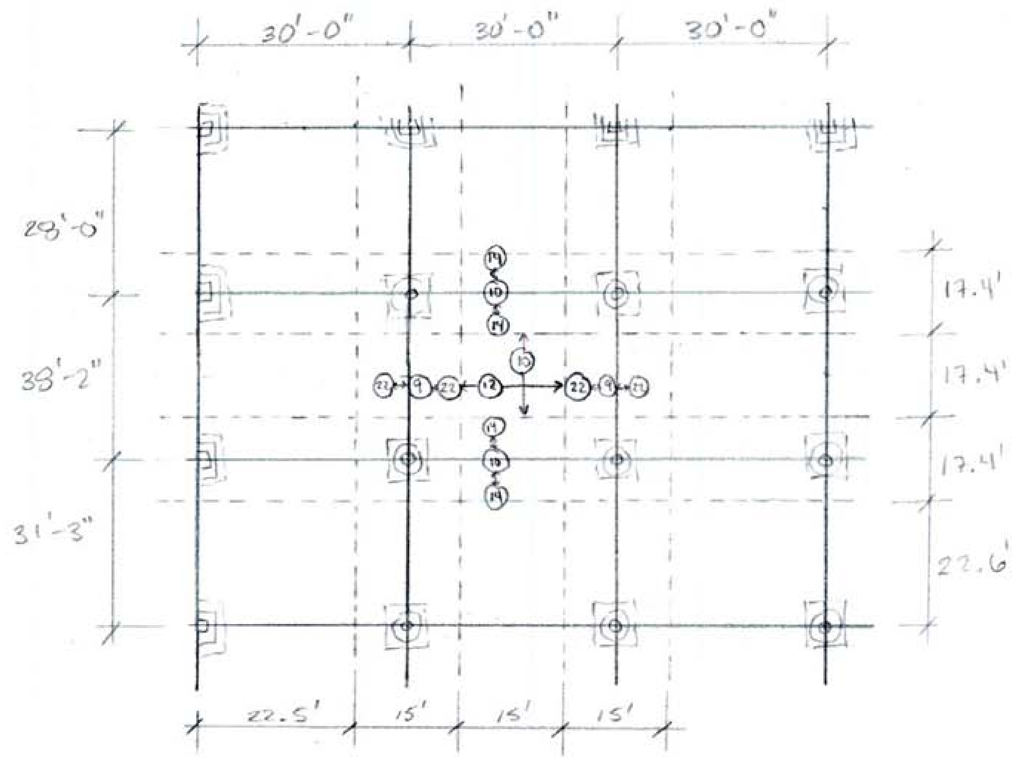
$t_{DROP} = 3''$ $WIDTH_{DROP} = 12.75'$

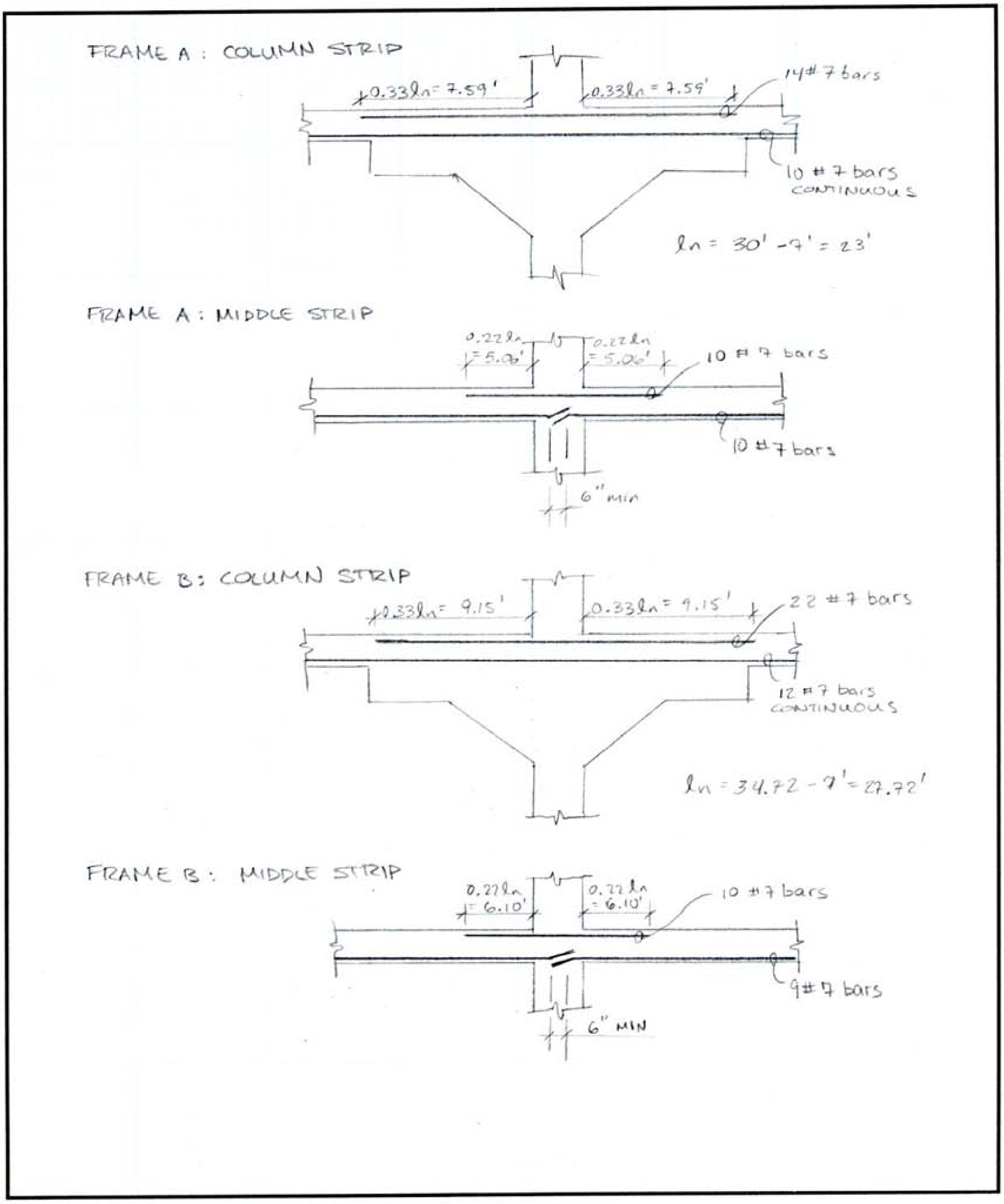
NOTE: ALL REBAR #7 BARS

QUANTITY OF BARS IN SECTION INDICATED IN CIRCLE

$f'_c = 5000$ psi

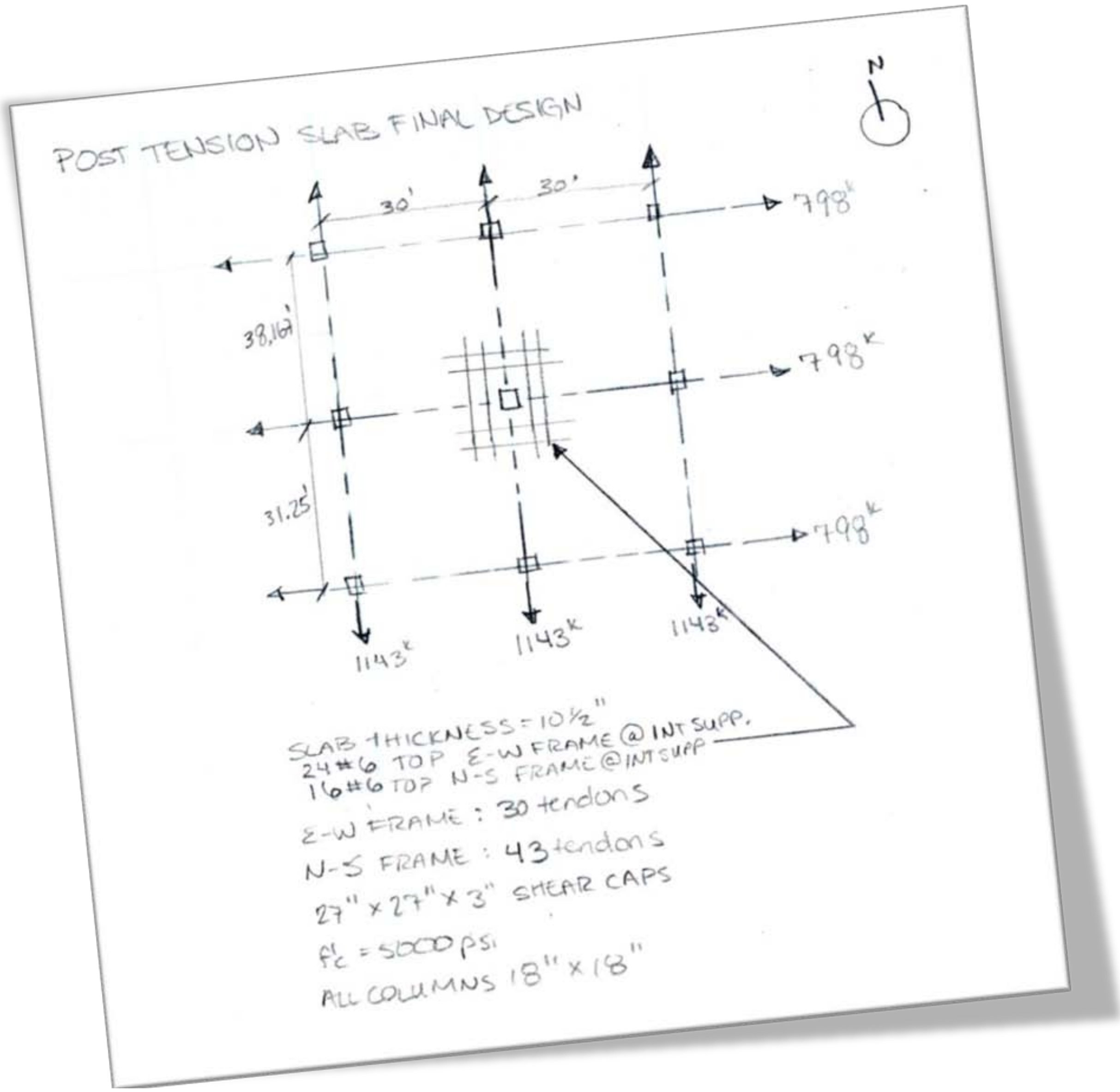
ALL COLUMNS 18" x 18"




FRAME A : COLUMN STRIP
0.33ln = 7.59'
0.33ln = 7.59'
14 # 7 bars
10 # 7 bars CONTINUOUS
ln = 30' - 7' = 23'
FRAME A : MIDDLE STRIP
0.22ln = 5.06'
0.22ln = 5.06'
10 # 7 bars
10 # 7 bars
6" min
FRAME B : COLUMN STRIP
0.33ln = 9.15'
0.33ln = 9.15'
22 # 7 bars
12 # 7 bars CONTINUOUS
ln = 34.72 - 7' = 27.72'
FRAME B : MIDDLE STRIP
0.22ln = 6.10'
0.22ln = 6.10'
10 # 7 bars
9 # 7 bars
6" MIN

System 3: Two-way post-tensioned Concrete Slab

This floor system consists of a two-way post-tensioned concrete slab. A typical interior bay was analyzed and designed for this section resulting in a 10.5 inch thick slab with (30) ½" diameter 270 ksi 7-wire strands in the east/west direction and (43) in the north/south direction. Minimum reinforcement was provided at midspan, while negative moment reinforcement at the supports was determined by ultimate strength requirements. The slab did not meet punching shear requirements due to the heavy loadings, to offset this, a 3" thick shear car on the 18"x18" columns. Shown below is the final design for the two-way post tensioned slab floor system.



Based on the required force to counteract the load in the interior bay, the number of tendons needed for the east/west and north/south frame was 23 and 22 respectively. However, these did not provide enough strength immediately after jacking or at service loads. The system was found to work at both stages when the number of tendons was increased to 30 and 43 for the east/west and north/south directions respectively.

Pro-Con Analysis: Two-Way Post-Tensioned Floor System

This system is very efficient when spanning great distances and carrying heavy loads. Structural sandwich of the floor system is the smallest for this system than any of the other alternate systems considered. Larger spans reduce the amount of columns in the building, creating larger open spaces which are important in the circulation areas as well as assembly halls. Large open spaces and thin structural sandwich help the building achieve its architectural goal of creating a transparent and welcoming appearance from the exterior.

If this floor system would be implemented into the design of Hunter College School of Social Work, the lateral systems would need to be changed from the existing systems. Construction for this system is very difficult and requires an experienced construction team. Most penetrations must be planned prior to construction to avoid coring through post-tensioning strands. This system is also dangerous since the pre-stressed tendons hold a large amount of tension, if snapped, could cause serious injury.

The self weight of this floor system is greater than all the systems included the existing system. It weights 131psf and is closely followed by the two-way flat slab with drop panels, weighing in at 130psf. This weight which is 130% larger than the existing system is not viable due to the limitation on crane size and capacity. These limitations are due to the site location which receives major traffic in the heart of East Harlem. If a larger crane was to be used, 119^{th} street would need to be shut down and that is not an option.

Figure10. Cut-out of post-tensioned slab

System 4: Precast hollow core plank on steel beam

Pre-cast hollow core planks were studied for their ability to span long distances, while maintaining a light self weight. Hollow core planks were sized according to Nitterhouse Concrete Products (on next page). A 10" thick x 4' wide hollow core plank spanning 28'-0" was determined to be adequate for service dead load and live load of the structure. A lightweight concrete topping 2" thick provides some fire resistance as well as rigidity to the floor system so that it acts as a rigid diaphragm to reduce lateral displacements due to lateral loading. The planks by themselves have a 2 hour fire rating without the need of additional fire proofing.

Steel beams were chosen to support these planks due to their lower self weight and because it reduces the need to redesign the lateral force resisting system and its location. The braced frame trusses would attach to the steel beams, which were found to be w24x76 based on required strength.

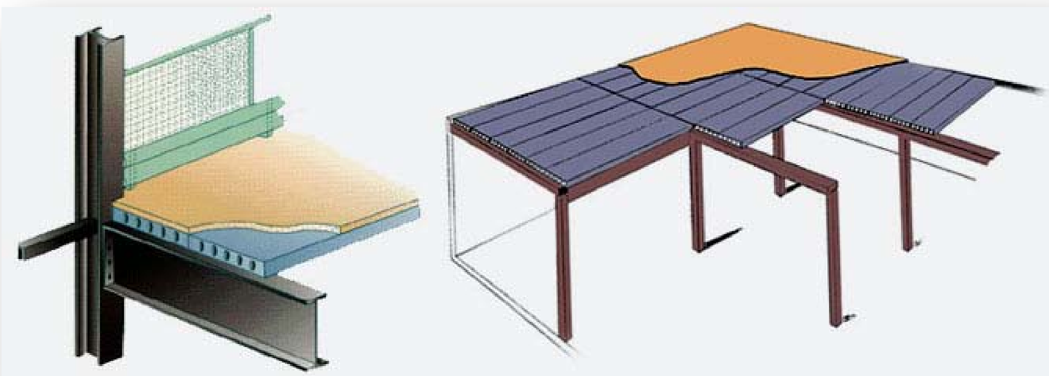


Figure 11. Precast hollow core plank on steel beam floor system

<u>Pro-Con Analysis: Pre-Cast Hollow Core Plank on Steel Beam Floor System</u>

The main advantage of using the pre-cast hollow core plank system is its production efficiency and ease of availability. Members are prefabricated in a pre-cast plant, ensuring a higher quality product and reducing site construction time since it's not cast-in-place and you don't need to wait for the concrete to cure. Therefore, construction is simple any time of the year regardless of temperature and humidity conditions. Pre-cast planks already meet the required two hours fire ratings so there is no need for additional fireproofing materials.

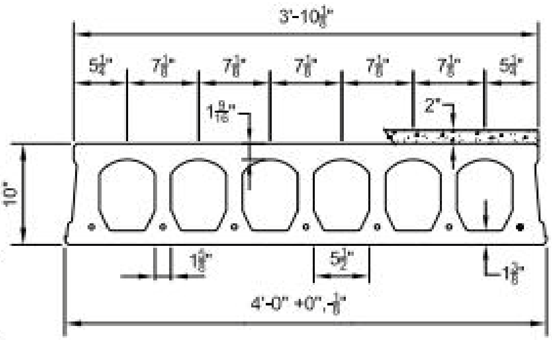
Hollow core planks contain less material than traditional concrete slab floor systems, which makes it have the second lowest self weight of the systems considered, only surpassed by the existing steel frame system. At 71 psf of self weight it is a viable alternative since it will not require a large crane. By using steel beams to support the planks, the existing braced frames can still be used as part of the lateral load resisting design.

Prestressed Concrete
10"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 327 in.^{2}	Precast b_w = 13.13 in.
I_c = 5102 in.^{4}	Precast S_{bcp} = 824 in.^{3}
Y_{bcp} = 6.19 in.	Topping S_{ttc} = 1242 in.^{3}
Y_{tcp} = 3.81 in.	Precast S_{tcp} = 1340 in.^{3}
Y_{ttc} = 5.81 in.	Precast Wt. = 272 PLF
	Precast Wt. = 68.00 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 168.1 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 191.7 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS																				IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)
Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	202	181	161	144	128	114	101	90	79	69	60	52	45	38					
7 - 1/2"ø	LOAD (PSF)	246	222	200	180	162	146	131	118	105	94	84	74	66	58					

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

11/03/08

10F2.0T

Results

Criterion	Floor System Comparison - Typical Interior Bay				
	Existing Composite Steel	Existing Non Composite Steel	Two-Way Flat Slab w/ Drop Panels	Two-Way Post Tensioned Slab	Pre-Cast Hollow Core Planks on Steel Beams
self weight (psf)	57.3	57.3	130	131	71
slab depth (in.)	6.25	6.25	10.5	10.3	10
Total Depth (in.)	24.50	24.50	13.5	13.5	34.2
Constructability	Medium	Medium	Medium	Hard	Easy
Foundation Impact	n/a	n/a	Major	Yes	Yes
Architectural Impact	n/a	n/a	Major	No	No
Transfer System Impact	n/a	n/a	Major	Major	Yes
Lateral System Impact	n/a	n/a	Yes	Yes	No
Vibration	Average	Average	Best	Above Average	Average
Fire Rating (hr)	2	2	2	2	2
Total Cost per ft^2 (S)	32.43	46.02	26.13	29.69	36.72
Possible Alternative	n/a	n/a	No	Yes	Yes

Figure 12. Floor system comparison for an interior bay

Comparison of Systems

After completing a side by side comparison of each schematic design, it is seen that the three alternative systems chosen for analysis are very economical in comparison with the existing system with the exception of pre-cast hollow core planks on steel beams. Pre-cast hollow core planks however, was the only one of the alternative systems that would be viable since it is light enough for the crane size specified and simple enough to construct under the space limitations of the project site. Due to their high self-weight, two-way flat slab and the two-way post-tensioned slab are out-of-the question and cannot be used.

Price comparison between the only viable alternative option; hollow core planks, with the existing steel frame floor system shows than they are fairly similar. The existing framing system consists of composite ($32.43/sqft) as well as non-composite beams ($46.02/sqft); compared to the price of hollow core planks floor system ($36.72/sqft) it is not apparent which is more economical. A more in-depth study must be done to determine cost gains or losses.

Conclusion

Technical Report II examines alternative floor systems in efforts to discover a system that is a viable option for use within Hunter College School of Social Work. All systems were chosen with careful consideration to the reduction of floor thickness, self weight, and its ability to span large bay sizes. Increasing the bay size was not explored since doing so would increase steel member size, which could increase the size of the crane….adding money to the project, and could cause an issue with lane closure in the street. The steel erector will be using a Manitowac 888 crane and anything bigger than this model cannot be used as Turner Construction Company cannot shut 119th street down.

Ignoring the total slab depth criteria, the best alternative option is the hollow core planks on steel beams system. It is one of the most economical and constructible system in this study. It is so economical due to the low labor costs for floor system erection and because they are pre fabricated off-site using less material than traditional concrete beams. A self weight of 71 psf would lead to increasing member sizes for the transfer systems and possible mat foundation redesign, but this may still be economically feasible due to less steel members being used (no infill beams). This floor system also has the ability to utilize a braced frame to resist lateral forces and can span great distances. Therefore, hollow core planks on steel beams are worth considering pending a more in-depth economic study.

Appendix A - Calculations

System 1: Composite steel beam and deck floor system (existing)

Figure A-1: Accumulated Loads on Columns

LOCATION J3 : Accumulated Loads on Columns											
Level	tributary area	dead load (psf)	live load (psf)	influence area	LL red. Factor	live load (k)	dead load (k)	load comb.	load at floor (k)	accum. Load (k)	accum. load (k) by Turner
roof	525	90	45	2100	1.00	23.6	47.3	1.2D+0.5Lr	68.5	68.5	80
Eighth	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	161.7	161
seventh	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	255.0	242
sixth	525	71	100	2100	0.58	30.3	37.3	1.2D+1.6L	93.2	348.2	337
fifth	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	460.4	715
fourth	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	572.6	852
third	675	71	100	3420	0.51	34.2	47.9	1.2D+1.6L	112.2	684.8	997
second	675	85	100	3420	0.51	34.2	57.4	1.2D+1.6L	123.6	808.4	1123
Ground	675	130	100	3420	0.51	34.2	87.8	1.2D+1.6L	160.0	968.4	1349

At level 5 there is a large difference between the accumulated loads calculated by that which was provided by Turner Construction Company. This is due to the step- back of the floor levels above. Since the columns located at J1.6 at above levels don't continue to the fifth level, the fifth level is forced to carry the load from the J1.6 column at level 6. Below is a table depicting the adjusted accumulated loads and how they compare to values provided by Turner Construction Company.

Figure A-2: Adjustment of Accumulated Loads on Columns

	accumulated load (k) by Turner for Loc. J1.6	LOCATION J3 : Accumulated Loads on Columns		
Level		Adjusted accumulated load (k)	accumulated load (k) provided by Turner	percent Error = |adj-prov| /adj*100
roof	n/a	68.5	80	17
eighth	n/a	161.7	161	0
seventh	n/a	255.0	242	5
sixth	266	348.2	337	3
fifth	n/a	726.4	715	2
fourth	n/a	838.6	852	2
third	n/a	950.8	997	5
second	n/a	1074.4	1123	5
Ground	n/a	1234.4	1349	9

COLUMN SPOT CHECK

COLUMN AT ROOF LEVEL @ J3 : W14×90

$P_u = 68.5^k$ (SEE SPREADSHEET)

$h = 14ft - 4ft = 10ft$ (SPLICE HT)

$A_g = 26.5\ in^2$

$I_x = 999\ in^4$

$I_y = 362\ in^4$

$r_x = 6.14$

$r_y = 3.70$

$A_T = (24.96)(21.04) = 525\ ft^2$

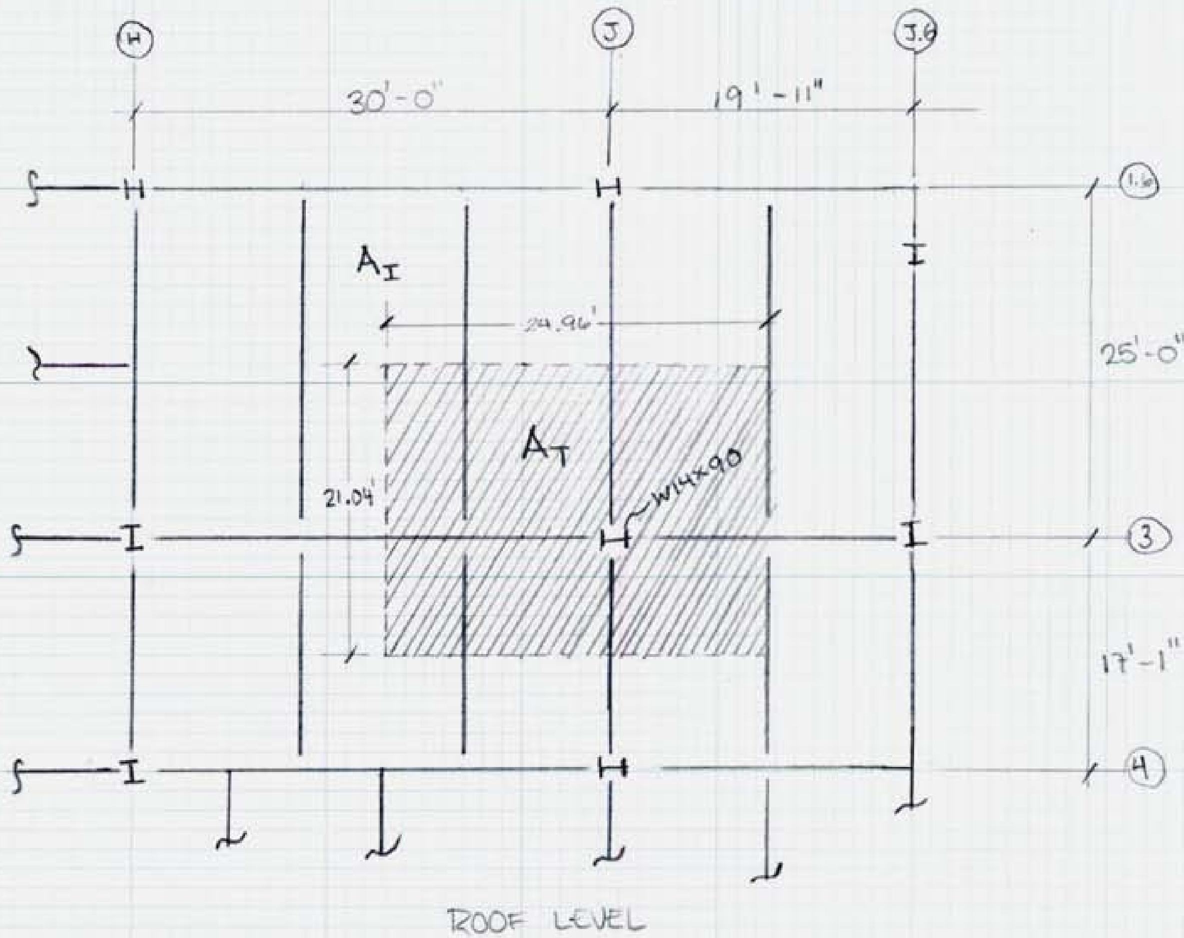
$A_I = (49.92)(42.08) = 2100\ ft^2$

$\frac{KL}{r_x} = \frac{10(12)}{6.14} = 19.5$

* CONTROLS $\frac{KL}{r_y} = \frac{10(12)}{3.70} = 32.4$

$\frac{KL}{r} \le 4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113 > 32.4$

∴ INELASTIC BEHAVIOR



$$f_{CR} = \left[0.658^{F_y/F_e}\right] F_y = \left[0.658^{50/273}\right](50) = 46.3 \text{ ksi}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29000)}{(32.4)^2} = 273 \text{ ksi}$$

$$\phi P_n = \phi F_{CR} A_g = (46.3)(0.9)(26.5) = 1104^k$$

$$P_u = 68.5^k << \phi P_n = 1104^k$$

CHECK W/ TABLE 4-22:

$\frac{KL}{r} = 32.4$ $\quad \phi f_{CR} = 41.7$ ksi

$f_{CR} = \frac{41.7}{\phi} = \frac{41.7}{0.9} = 46.3$ ✓ METHOD BY HAND CHECKS

CHECK W/ TABLE 4-1:

$KL = 10'$ $\quad$ W14×90

$\phi P_n = 1100^k \approx 1104^k$ ✓ METHOD BY HAND CHECKS

*NOTE: TABLE 4-1 SHALL BE USED FOR REMAINING COLUMN CHECKS AS IT IS BASED ON METHOD BY HAND SHOWN ABOVE

COMMENTS: COLUMN SIZES ARE VERY LARGE WHILE CONSIDERING GRAVITY LOADS ALONE, HOWEVER, IT IS PART OF A MOMENT CONNECTION WITH FRAME 7 & TRUSS 9 (see Appendix C for Braced Frames) HENCE IT RECEIVES LARGE INDUCED MOMENTS

LEVEL 8: $P_u = 161.7^k$
W14×90
$h = 10$ ft $= KL$
TABLE 4-1: $\phi P_n = 1100^k > P_u = 161.7$ ✓ OK

LEVEL 7: $P_u = 255^k$
W14×233
$h = 26$ ft $= KL$
TABLE 4-1: $\phi P_n = 2020^k > P_u = 255$ ✓ OK

LEVEL 6: $P_u = 348.2^k$
W14×233
$h = 26$ ft $= KL$
TABLE 4-1: $\phi P_n = 2020^k > 348^k = P_u$ ✓ OK

LEVEL 5: $P_u = 762.4^k$
W14×311
$h = 28$ ft $= KL$
TABLE 4-1: $\phi P_n = 2580^k > 762 = P_u$ ✓ OK

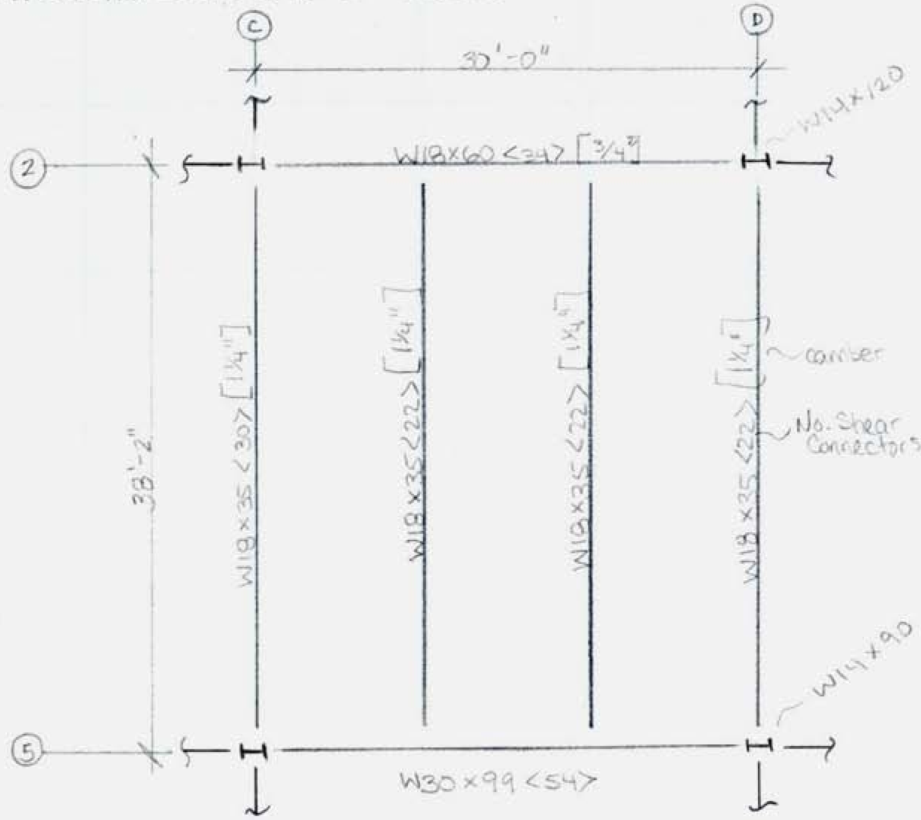
LEVEL 4: $P_u = 838.6^k$
W14×311
$h = 28$ ft $= KL$
TABLE 4-1: $\phi P_n = 2580^k > 838$ ✓ OK

LEVEL 3: $P_u = 950.8^k$
W14×550
$h = 31$ ft $= KL$
TABLE 4-1: $\phi P_n = 4350^k > 950.8$ ✓ OK

LEVEL 2: $P_u = 1074.4^k$
W14×550
$h = 31$ ft $= KL$
TABLE 4-1: $\phi P_n = 4350^k > 1074$ ✓ OK

GROUND LEVEL: $P_u = 1234.4^k$
W14×730
$h = 31$ ft $= KL$
TABLE 4-1: $\phi P_n = 6150^k > 1234$ ✓ OK

TYPICAL INTERIOR BAY : 2nd FLOOR



LOADS

LIVE LOADS: 40 psf & 100 psf

$LL\ Red = 0.25 + \frac{15}{\sqrt{763.3}} = 0.79 \Rightarrow LL = 79\ psf$

DEAD LOAD = 71 psf

(LOADING AREAS 8 & 9)

FLOOR SYSTEM: 3 1/4" LT. WT. CONC. OVER 3" - 18 GAGE METAL DECK

LOAD COMBINATION: 1.2D + 1.6L

BEAM SPOT CHECK

FACTORED LOAD: 1.2D+1.6L

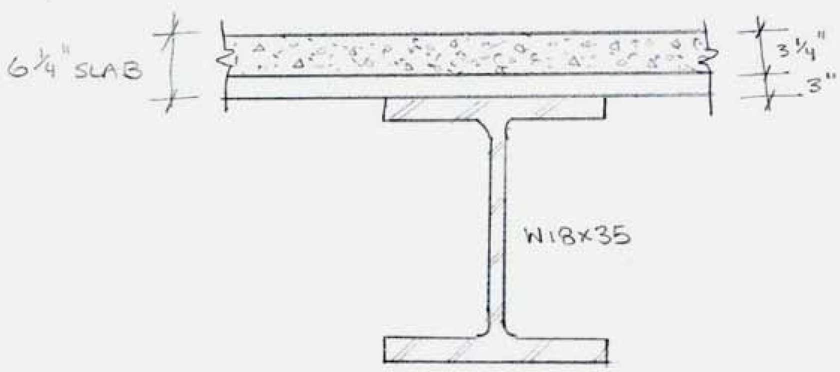
$W_u = 1.2(71) + 1.6(79) = 211.6$ psf

$\therefore$ TRIB WIDTH = 10'

$W_u = 211.6\text{ psf}(10')/1000 = 2.116$ klf

$$M_u = \frac{W_u \ell^2}{8} = \frac{(2.116)(38.167)^2}{8} = 385^{'k}$$

$$b_{eff} = \min \begin{cases} \text{SPACING} = 10' \times 12 = 120'' \\ \dfrac{\text{SPAN}}{4} = \dfrac{(38.167')(12''/ft)}{4} = 114.5'' \leftarrow \text{CONTROLS} \end{cases}$$



CHECK FOR DEFLECTION UNDER CONSTRUCTION LOADS:

$$\Delta_{CONST} = \frac{5 W_{CONC} \ell^4}{384 IE}$$

$W_{CONC} = 110\text{ pcf}\left(3.25''/12\right) = 29.8$ psf

$W_{CONC} = 29.8\text{ psf}(10') = 298\text{ plf} = 0.298$ klf

$$\Delta_{allow} = \ell/360 = \frac{38.167(12)}{360} = 1.27''$$

$$I_{req} = \frac{5 W_{conc} \ell^4}{384 \Delta_{const} E} = \frac{5(0.298)(38.167)^4(1728)}{384(1.27)(29000)} = 386\text{ in}^4$$

$$I_{W18\times35} = \underbrace{510\text{ in}^4 > 386\text{ in}^4}_{} \therefore \text{OK}$$

LARGE DIFF. B/C DESIGN NOT CONTROLLED BY DEFL. UNDER CONST. LOAD.

CHECK BENDING FOR CONSTRUCTION LOADING:

$w_{conc} = 0.298$ klf

$w_{LIVE} = 20(10) = 0.200$ klf

$w_u = 1.2(0.298) + 1.6(0.200) = 0.678$ klf

$M_u = \frac{w_u l^2}{8} = \frac{0.678(38.167)^2}{8} = 123^{k} < \phi M_n = 249^{k}$ ∴ OK

FROM TABLE 3-19:

ASSUME $a = 1$ ∴ $Y2 = 5\frac{3}{4}''$ → PNA = 6"

TRY W18×38 LOCATION 6 : $\Sigma Q_n = 194$, $\phi M_p = 419 > M_u = 385$ ∴ OK

$b_{eff} = 114.5''$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{194}{0.85(3.5)(114.5)} = 0.57 < 1.0$ ∴ OK CONSERV.

$Y2 = 6.25 - \left(\frac{0.57}{2}\right) = 5.96 \Rightarrow \phi M_p = 418 > M_u$ ∴ OK

CHECK NUMBER OF SHEAR STUDS REQUIRED: TABLE 3-21

STUD DIAM. = 3/4"
DECK PERPENDICULAR
LIGHT WT. CONC
$f'_c = 3$ ksi (CONSER.)
$R_p = 0.6$

FIND Q_n

$\#\text{STUDS}_{REQ'D} = \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{194}{17.2} \times 2 = 22.5$ req [1 STUD/RIB]

$\#\text{STUDS}_{PROVIDED} = 22$ [STUDS PLACED @ 20" O.C. OVER LENGTH OF 38'-2"] ∴ OK

CHECK DEFLECTION: - TABLE 3-20:

$Y2 = 5.5'' \Rightarrow I_{LB} = 1220$ in^4

$\Delta = \frac{5 w_u l^4}{384 E I_{LB}} = \frac{5(0.79)(38.167)^4(1728)}{384(29000)(1220)} = 1.066''$

$w_u = (79 \text{ psf})(10')/1000 = 0.79$ klf

$\Delta_{allow} = \frac{l}{360} = \frac{38.167 \times 12}{360} = 1.27''$

NOTE: THE BEAM HAS A $1\frac{1}{4}''$ CAMBER ∴ $\Delta_{ACT} = 1.25 - 1.066$

$\Delta_{ACT} = 0.18'' < 1.27'' = \Delta_{allow}$ ∴ OK

GIRDER SPOT CHECK

LOADS:

DEAD LOAD = 71 psf

$$LL Red = 0.25 + \frac{15}{\sqrt{1145 \times 2}} = 0.56$$

LIVE LOAD = 56 psf

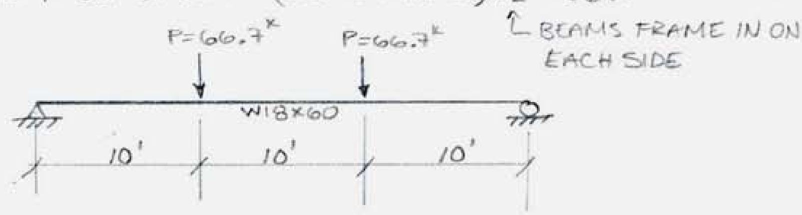
$$w_{uDL} = 1.2(71)(10')/1000 = 0.852 \text{ klf}$$

$$P_{DL} = \frac{w_{uDL}\,l}{2} = \frac{0.852\,(38.167)}{2} = 16.26^{k}$$

$$w_{uLL} = 1.6(56)(10')/1000 = 0.892 \text{ klf}$$

$$P_{LL} = \frac{w_{uLL}\,l}{2} = \frac{0.892\,(38.167)}{2} = 17.10^{k}$$

TOTAL P ON GIRDER = $(16.26 + 17.10) \times 2 = 66.7^{k}$



$$M_{max} = 66.7'(10') = 667^{1k}$$

ASSUME $a = 1$: $Y2 = 5\tfrac{3}{4}'' \rightarrow 6''$

FOR W18×60, LOC. 7 : $\Sigma Q_n = 220$, $\phi M_p = 666^{1k}$

$b_{eff} = 90''$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{220}{0.85(3.5)(90)} = 0.822''$$

$$Y2 = 6.25'' - \left(\frac{0.822''}{2}\right) = 5.84 \Longrightarrow \phi M_p = 661^{1k} < M_u \therefore NG$$

TRY LOC. 6 : $\Sigma Q_n = 288$, $\phi M_p = 712^{1k}$

$$a = \frac{288}{0.85(3.5)(90)} = 1.075 \therefore \text{ ASSUMP. } a=1 \text{ not valid}$$

$$Y2 = 6.25 - \left(\frac{1.075}{2}\right) = 5.71 \longrightarrow \phi M_p = 703^{1k} > M_u$$

ASSUME $a = 2$ $\therefore$ $Y2 = 5.25'' \rightarrow 5.5''$

$a = 1.075 < 2 \therefore OK$ LOC 6 : $\phi M_p = 701^{1k} > M_u = 667^{1k}$

No. OF SHEAR STUDS REQ'D : $\frac{288}{17.2} \times 2 = 34$ STUDS $\therefore$ OK

PROVIDED = 34 STUDS ✓

GIRDER SPOT CHECK CONTINUED:

CHECK DEFLECTION

$I_{LB} = 2620 \text{ in}^4$ [TABLE 3-20]

$$\Delta = \frac{5 w_u l^4}{384 E I_{LB}}$$

$$\Delta = \frac{5(0.298)(30)^4(1728)}{384(29000)(2620)} = 0.07''$$

$w_u = 29.8 \text{ psf } (10')/1000 = 0.298 \text{ klf}$

$$\Delta_{allow} = l/360 = \frac{30(12)}{360} = 1''$$

$\Delta_{allow} = 1'' > \Delta_{ACT} = 0.07''$ ∴ OK

System 2: Two-way reinforced concrete slab with drop panels and flared column capitals

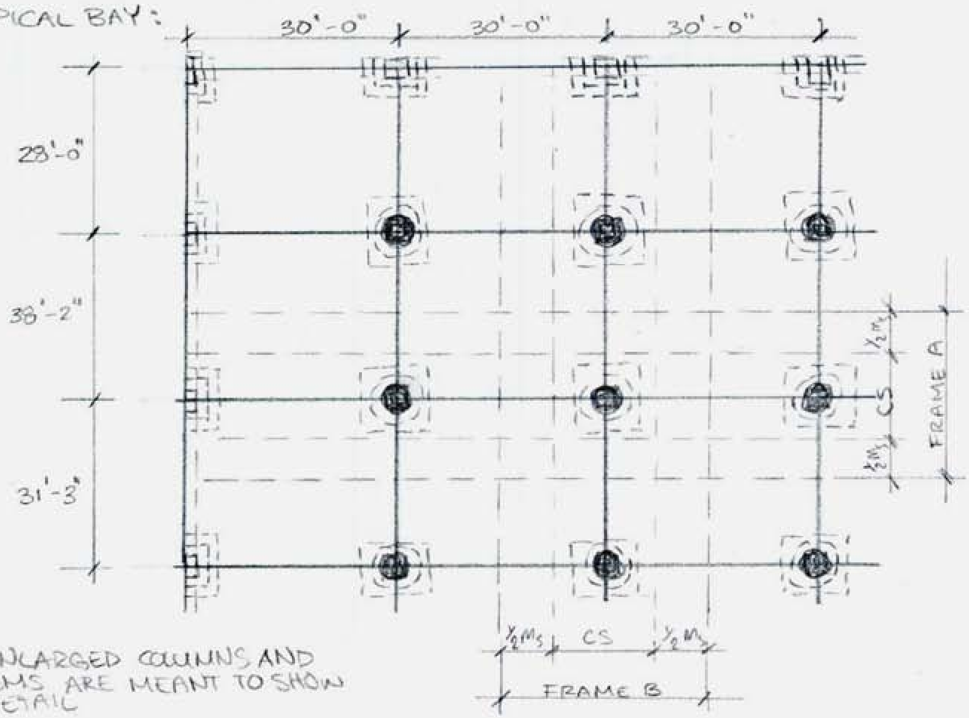
TWO WAY FLAT SLAB W/DROP PANELS

ASSUMPTIONS: f'_c = 5000 psi
f_y = 60000 psi
ω_c = 150 pcf
LIVE LOAD = 48 psf
SUPERIMPOSED DEAD LOAD = 20 psf

ASSUME 18"x18" COLS

LL Red = 0.48L_o

TYPICAL BAY:



NTS: ENLARGED COLUMNS AND BMS ARE MEANT TO SHOW DETAIL

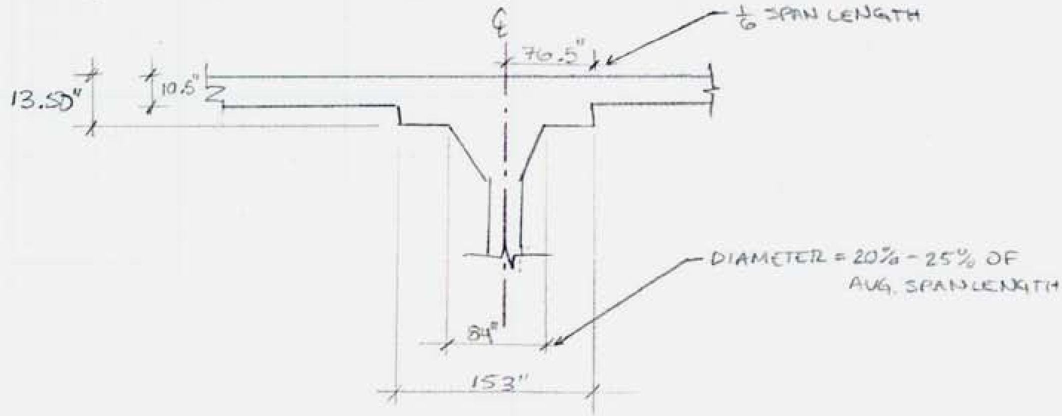
min. slab thickness based on deflection: (ACI 9.5.3.3 TABLE 9.5(c))

$t_{min} = \ell_n/36$

[MUST SIZE DROP PANEL + COL CAPITOLS TO OBTAIN ℓ_n; ℓ_n = FACE-TO-FACE OF COL. CAPITOLS]

TABLE 9.5C GIVES THICKNESS LIMITATIONS BASED UPON DEFLECTION CRITERIA. THEREFORE DEFLECTION IS OK BY INSPECTION.

SIZE COL. CAPITOLS + DROP PANELS: (ACI 13.2.5)



$\frac{1}{6}$ SPAN $= \frac{1}{6}(38.167 \times 12) = 76.3"$ USE 76.5" SO TOTAL DROP WIDTH $= 12.75' = 153"$

DIAM. OF COL CAPITAL $= 0.20\left(\frac{38.167 + 31.25}{2}\right) = 6.94' = 83.3"$ SAY $7' = 84"$

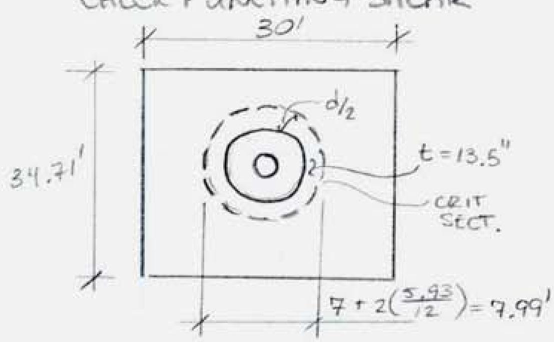
$l_n = 38.167 - 6.94 = 31.227' = 375"$

$t_{slab} = \frac{l_n}{36} = \frac{375}{36} = 10.42"$ ∴ USE 10.50"

$t_{drop} = \frac{1}{4} t_{slab} = \frac{1}{4}(10.5) = 2.625"$ ∴ USE 3"

} 13.50" THK @ DROP PANEL

CHECK PUNCHING SHEAR



$d_{long} = 13.5" - \frac{3}{4} - \frac{1}{2}(0.875) = 12.3125 = 12.3"$

$d_{short} = 12.3" - 0.875 = 11.4"$

$d_{avg} = \frac{12.3 + 11.4}{2} = 11.85"$

$d_{avg}/2 = 5.93"$

PERIMETER b_o:

$b_o = \pi(7.99 \times 12) = 301"$

$b_o/d = \frac{301}{11.85} = 25.4$

$\beta_c = 1.0$ for circ col.

$\alpha_s = 40$ for int col.

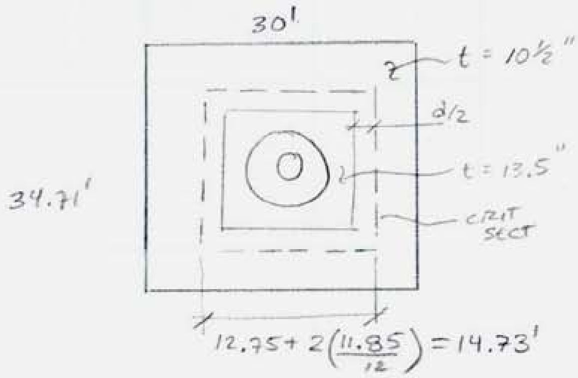
$w_u = 1.2\left(\frac{10.5}{12}(150 + 71)\right) + 1.6(48) = 309\,psf$

$V_u = 0.309\left(30 \times 34.71 - \pi(7.99)^2/4\right) = 306^k$

$$V_c = \min \text{ of } \begin{cases} 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}(301)(11.85)/1000 = 1009^k \\ \left(2 + 4/\beta_c\right)\sqrt{f'_c}\, b_o d = (2+4)\sqrt{5000}(301)(11.85)/1000 = 1513^k \\ \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40}{25.4} + 2\right)\sqrt{5000}(301)(11.85)/1000 = 902^k \leftarrow \text{CONTROLS} \end{cases}$$

$\phi V_c = 0.75(902^k) = 670^k > V_u = 306^k$ ∴ OK

CHECK PUNCHING SHEAR AROUND DROPS :



$d_{AVG} = 11.85''$

Perimeter b_o :

$b_o = 14.73(4)(12) = 707''$

$b_o/d = 707/11.85 = 59.6''$

$V_u = 0.309\,(30 \times 34.71 - 14.73(14.73)) = 255^k$

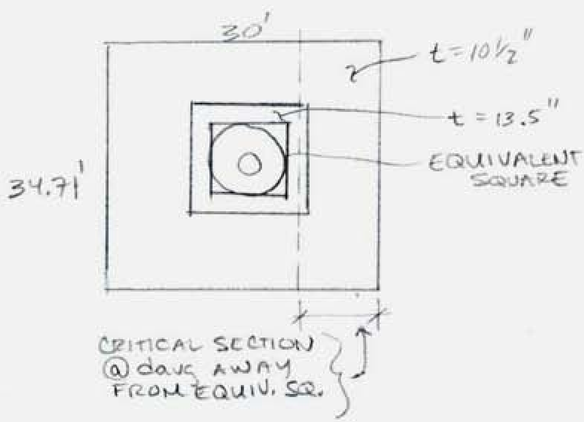
$\beta_c = 1.0$ for circ. cols.

$\alpha_s = 40$ for int. cols.

$$V_c = \min \text{of} \begin{cases} 4\sqrt{f'_c}\, b_o = 4\sqrt{5000}\,(707)(11.85)/1000 = 2369.6^k \\ (2 + 4/\beta_c)\sqrt{f'_c}\, b_o d = (2+4)\sqrt{5000}\,(707)(11.85)/1000 = 3554^k \\ \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40}{59.6} + 2\right)\sqrt{5000}\,(707)(11.85)/1000 = 1582^k \leftarrow \text{CONTROLS} \end{cases}$$

$\phi V_c = 0.75(1582) = 1187^k > V_u = 255^k \quad \therefore$ OK

CHECK WIDE BEAM ACTION :



$d_{avg} = 11.85''$

diam of EQUIV SQ $= a^2$

$a^2 = \frac{\pi (7')^2}{4} = 38.5'$

$a = 6.2'$

CRITICAL SECTION :

$\frac{30'}{2} - \frac{6.2'}{2} - \frac{11.85}{2(12)} = 11.4'$

$V_u = 0.309\,\text{ksf}\,(11.4')(34.71)$

$= 122.3$ kips

$V_c = 2\sqrt{f'_c}\; b_w d = 2\sqrt{5000}\,(30 \times 12)(11.85 - 3)/1000 = 451^k$

$\phi V_c = 0.75\,(451) = 338^k > V_u = 122.3^k \quad \therefore$ OK

FIND MOMENTS AT CRITICAL SECTIONS:

FRAME A: $M_o = \frac{1}{8} W_u l_2 l_1 \left(1 - \frac{2c}{3l_1}\right)^2$

$$W_u = 1.2\left[\frac{10.5}{12}(150) + 71\right] + 1.6(48) = 0.309 \text{ ksf}$$

$$M_o = \frac{1}{8}(0.309)(38.167)(30)^2\left(1 - \frac{2(7)}{3(30)}\right)^2 = 946^{1K}$$

FRAME B: $M_o = \frac{1}{8}(0.309)(30)(38.167)^2\left(1 - \frac{2(7)}{3(38.167)}\right)^2 = 1300^{1K}$

MOMENT DISTRIBUTION: (ACI 13.6.3.2) NO EDGE BMS

MOMENT	FRAME A	FRAME B	
M^-	614.9	845.0	$0.65 M_o$
M^+	331.1	455.0	$0.35 M_o$

ACI 13.6.4: NO EDGE BMS

$\alpha l_2 / l_1 = 0$

75% M^- to CS 25% M^- to MS

60% M^+ to CS 40% M^+ to MS

SUMMARY OF MOMENTS:

FRAME A: TOTAL WIDTH = 34.71'; CS = 17.4'; MS = 17.4'

TOTAL MOMENT	−614.9	+331.1	−614.9
CS SLAB	−461.2	+198.7	−461.2
MS SLAB	−153.7	+132.4	−153.7

FRAME B: TOTAL WIDTH = 30'; CS = 15'; MS = 15'

TOTAL MOMENT	−845.0	+455.0	−845.0
CS SLAB	−633.8	+273.0	−633.8
MS SLAB	−211.3	+182.0	−211.3

DETERMINE REINFORCING :

	ITEM	DESCRIPTION	FRAME A CS m^-	FRAME A CS m^+	FRAME A MS m^-	FRAME A MS m^+	FRAME B CS m^-	FRAME B CS m^+	FRAME B MS m^-	FRAME B MS m^+
	1	M_u (ft-k)	-461	+199	-154	+132	-634	+273	-211	+182
	2	SLAB WIDTH, b (in.)	209	209	209	209	180	180	180	180
	3	EFFECT. DEPTH, d (in)	12.3	9.6	9.6	9.6	11.4	8.7	8.7	8.7
	4	$M_n = M_u/\phi = M_u/0.9$	-512	+221	-171	+147	-704	+303	-234	+202
	5	$M_n \times 12/b$ (k-in/in)	-26	+11	-9	+8	-42	+18	-14	+12
	6	$R_n = (M_n/bd^2) \times 12000$	194	137	107	92	361	267	206	178
	7	ρ (INTERPOLATION TABLE A-5A)	0.0033	0.0023	0.0018	0.0016	0.0062	0.0046	0.0035	0.0030
	8	$A_s = \rho bd$ (in²)	8.48	4.61	3.61	3.21	12.72	7.20	5.48	4.70
	9	$A_{s\,min} = 0.0018bt$	3.95	3.95	3.95	3.95	3.40	3.40	3.40	3.40
#7 BARS	10	N = LARGER A_s/0.60	14	8	7	7	22	12	10	8
	11	$N_{min} = \frac{\text{WIDTH OF STRIP}}{2t}$	10	10	10	10	9	9	9	9

EFFECTIVE DEPTHS :

FRAME A CS — $d = d_{LONG} = 12.3''$ (for m^-)

$d = 10.5 - 0.75 - \frac{1}{2}(0.275) = 9.61''$ (for m^+)

FRAME A MS — $d = 9.6''$ (for m^- and m^+)

FRAME B CS — $d = 12.3 - 0.875 = 11.4''$ (for m^-)

$d = 9.6 - 0.875 = 8.7''$ (for m^+)

FRAME B MS — $d = 8.7''$ (for m^- and m^+)

FINAL DESIGN: $t_{slab} = 10.5"$ DIAM. COL CAPS = 7'

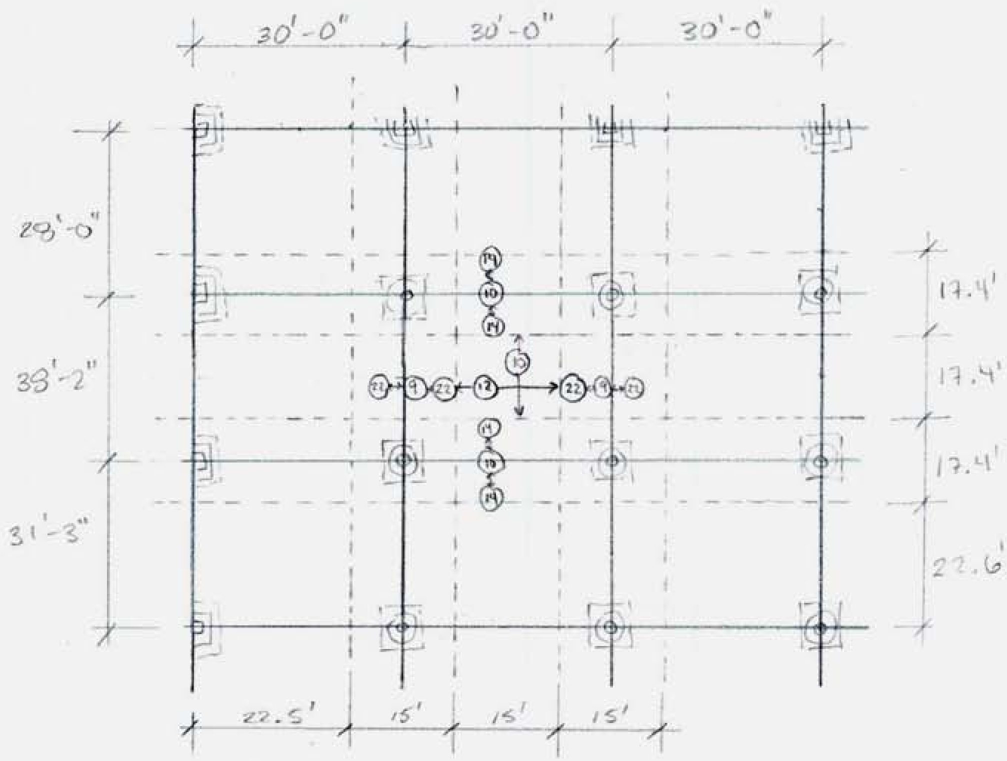
$t_{DROP} = 3"$ $WIDTH_{DROP} = 12.75'$

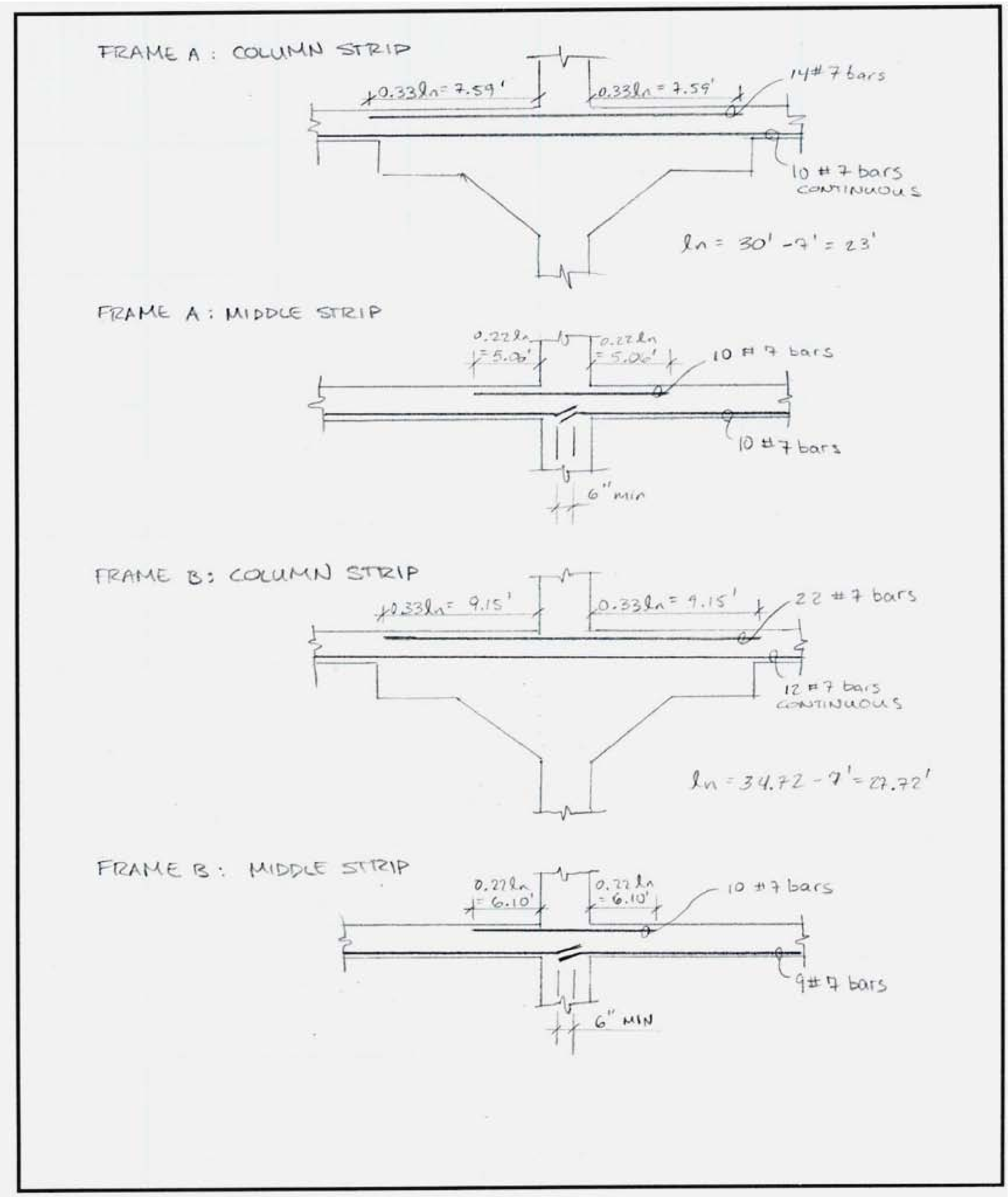
NOTE: ALL REBAR #7 BARS

QUANTITY OF BARS IN SECTION INDICATED IN CIRCLE

$f'_c = 5000$ psi

ALL COLUMNS 18" x 18"




FRAME A : COLUMN STRIP
0.33ln = 7.59'
0.33ln = 7.59'
14 #7 bars
10 #7 bars CONTINUOUS
ln = 30' - 7' = 23'
FRAME A : MIDDLE STRIP
0.22ln = 5.06'
0.22ln = 5.06'
10 #7 bars
10 #7 bars
6" min
FRAME B : COLUMN STRIP
0.33ln = 9.15'
0.33ln = 9.15'
22 #7 bars
12 #7 bars CONTINUOUS
ln = 34.72 - 7' = 27.72'
FRAME B : MIDDLE STRIP
0.22ln = 6.10'
0.22ln = 6.10'
10 #7 bars
9 #7 bars
6" MIN

System 3: Two – way post-tensioned conc. Slab

TWO-WAY POST TENSIONED SLAB

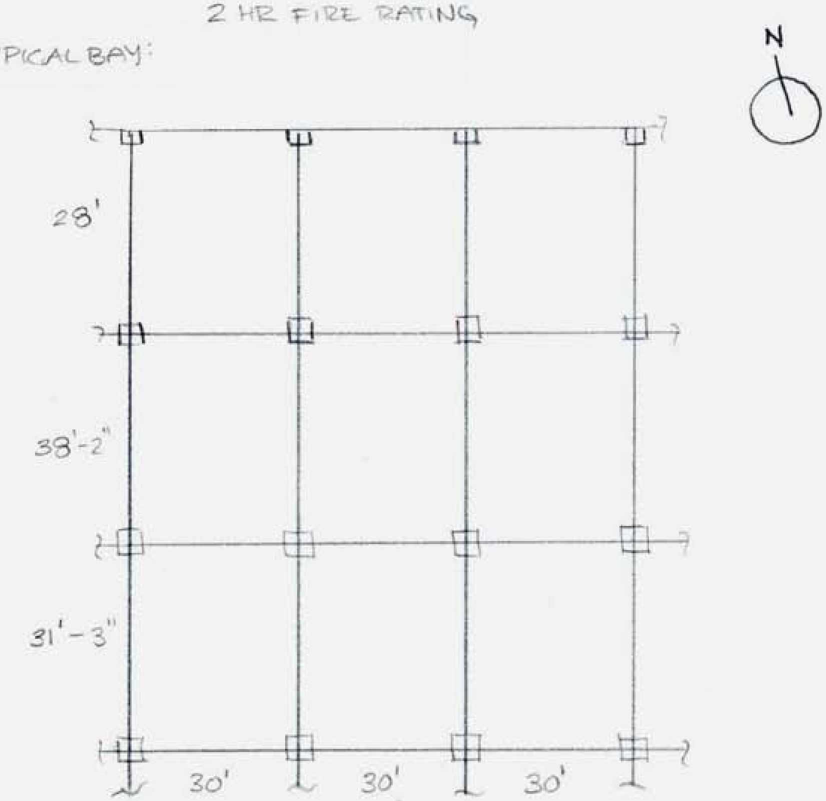
ASSUMPTIONS : $f'_c = 5000$ psi $f_y = 60000$ Assume 18" x 18" COLS

LIVE LOAD = 79 psf

DEAD LOAD = 71 psf

2 HR FIRE RATING

TYPICAL BAY:



ADDITIONAL MATERIALS:

$f'_{ci} = 3000$ psi

UNBONDED TENDONS — ½" Φ, 7-WIRE STRANDS,

$A = 0.153\ in^2$, $f_{pu} = 270$ ksi

ESTIMATED PRESTRESSED LOSSES = 15 ksi

$f_{se} = 0.7(270\ ksi) - 15\ ksi = 174\ ksi$

$P_{eff} = A(f_{se}) = 0.153\ in^2\ (174\ ksi) = 26.6\ ^K/\text{TENDON}$

DETERMINE PRELIMINARY SLAB THICKNESS:

$$\frac{\text{SPAN}}{\text{LENGTH}} = \frac{L}{45} = \frac{38.167(12)}{45} = 10.17'' \quad \text{TRY } 10\tfrac{1}{2}'' \text{ SLAB}$$

L: LARGEST SPAN

LOADING:

DEAD LOAD = 71 psf + 131 = 202 psf

SELF WEIGHT = 10.5" (150 pcf)/12 = 131 psf

LIVE LOAD = 79 psf

$1.2D + 1.6L = 1.2(71 + 131) + 1.6(79) = 369$ psf

DESIGN OF EAST - WEST INTERIOR FRAME

USE OF EQUIVALENT FRAME METHOD ACI 13.7 (EXCLUDING §13.7.7.4-5)

TOTAL BAY WIDTH BTWN. CENTERLINES = 38.167'

USED THE LARGEST BAY WIDTH (CONSERVATIVE)

IGNORE COL STIFFNESS IN EQUATIONS FOR SIMPLICITY OF HAND CALCS

NO PATTERN LOADING REQUIRED, SINCE $LL/_{DL} < 3/4$ (ACI 13.7.6)

CALCULATE SECTION PROPERTIES:

$A = bh = 38.167(12)(10\tfrac{1}{2}'') = 4809 \text{ in}^2$

$$S = \frac{bh^2}{6} = \frac{38.167(12)(10\tfrac{1}{2}'')^2}{6} = 8416 \text{ in}^3$$

SET DESIGN PARAMETERS:

ALLOWABLE STRESSES: CLASS U (ACI 18.3.3)

AT TIME OF JACKING (ACI 18.4.1)

$f'_c = 3000$ psi

COMPRESSION $= 0.60 f'_{ci} = 0.60(3000 \text{ psi}) = 1800$ psi

TENSION $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ psi

AT SERVICE LOADS

$f'_c = 5000$ psi

COMPRESSION $= 0.45 f'_c = 0.45(5000) = 2250$ psi

TENSION $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424$ psi

AVERAGE COMPRESSION LIMITS:

$P/A = 125$ psi min. (ACI 18.12.4)

$= 300$ psi max

TARGET LOAD BALANCES:

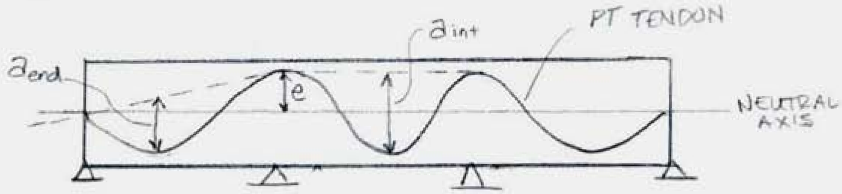
60% – 80% OF DL (SELF WT.) FOR SLABS (GOOD APPROX. FOR HAND CALCS)

TRY 75%: $0.75 w_{SELF} = 98$ psf

COVER REQUIREMENTS: 2 HR FIRE RATING

RESTRAINED SLABS = 3/4" BOTTOM

TENDON PROFILE:



TENDON ORDINATE	TENDON (CGC) LOC.
EXT. SUPPORT – ANCHOR	10.5/2 = 5.25"
INT. SUPPORT – TOP	10.5 – 1 = 9.5"
INT. SPAN – BOTTOM	1.0"
END SPAN – BOTTOM	1.75"

$a_{int} = 9.5" - 1" = 8.5"$

$a_{end} = (5.25" + 9.5")/2 - 1.75" = 5.625"$

PRESTRESSED FORCE REQ'D TO BALANCE 75% OF SELFWEIGHT DL

$w_b = 0.75\, w_{DL} = 98\,\text{psf}\,(38.167') = 3.740$ klf

$P = \dfrac{w_b L^2}{8 a_{int}} = \dfrac{3.740\,(30)^2}{8(8.5)/12} = 594^k$ = FORCE IN TENDONS TO COUNTERACT LOAD IN INTERIOR BAY.

CHECK PRECOMPRESSION ALLOWANCE:

USE 30 TENDONS

$P_{ACT} = 30\,(26.6\ ^k/\text{TENDON}) = 798^k$

ADJUST UNBALANCED LOAD

$w_b = \dfrac{798}{594}(3.740) = 5.024$ klf

DETERMINE ACTUAL PRECOMPRESSION STRESS

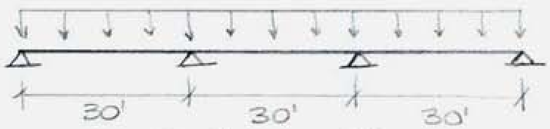
$$\frac{P_{ACT}}{A} = \frac{798(1000)}{10.5 \times 38.167 \times 12} = 166.94 \text{ psi} \quad \left.\begin{matrix} > 125 \text{ psi min} \\ < 300 \text{ psi max} \end{matrix}\right\} \therefore OK$$

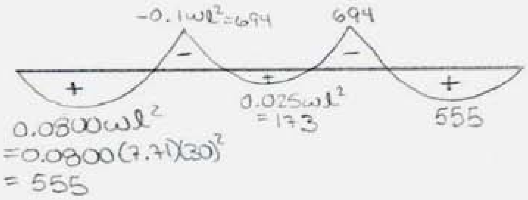
EFFECTIVE PRESTRESS FORCE = 798 FOR E-W FRAME

CHECK SLAB STRESSES:

DEAD LOAD MOMENTS

$w_{DL} = (38.167)(131 + 71) = 7.71$ klf

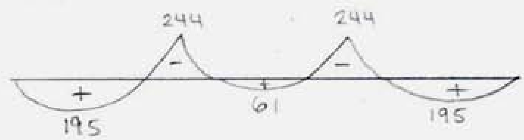




m (ft·k)

LIVE LOAD MOMENTS

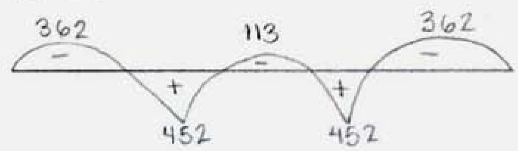
$w_{LL} = 38.167(71) = +2.710$ klf



m (ft·k)

TOTAL BALANCED MOMENTS

$w_b = -5.024$ klf



m (ft·k)

STAGE 1: STRESSES IMMED. AFTER JACKING (DL+PT)

MIDSPAN 2

$f_{TOP} = (-M_{DL} + M_b)/S - P/A$
$= 12000(-173 + 113)/8416 - 167 = -251$ psi < 1800 psi $\therefore$ OK

$f_{BOT} = (+M_{DL} - M_b)/S - P/A$
$= 12000(173 - 113)/8416 - 167 = -80$ psi $< 3\sqrt{f'_{ci}} = 164$ psi $\therefore$ OK

SUPPORTS

$f_{TOP} = (M_{DL} - M_b)/S - P/A$
$= (12000)(694 - 452)/8416 - 167 = 179$ psi $< 6\sqrt{f'_{ci}} = 329$ psi $\therefore$ OK

$f_{BOT} = (-M_{DL} + M_b)/S - P/A$
$= 12000(-694 + 452)/8416 - 167 = -511 < 0.6f'_{ci} = 1800$ psi $\therefore$ OK

STAGE 2: STRESSES @ SERVICE LOAD (DL + LL + PT)

MIDSPAN

$f_{TOP} = (-M_{DL} - M_{LL} + M_b)/S - P/A$

$= 12000(-173 - 61 + 113)/8416 - 167 = -338 \text{ psi} < 0.45 f'_c = 2250 \text{ psi} \therefore OK$

$f_{BOT} = (+M_{DL} + M_{LL} - M_b)/S - P/A$

$= 12000(173 + 61 - 113)/8416 - 167 = 6.5 \text{ psi} < 6\sqrt{f'_c} = 424 \text{ psi} \therefore OK$

SUPPORTS

$f_{TOP} = (+M_{DL} + M_{LL} - M_b)/S - P/A$

$= 12000(694 + 244 - 452)/8416 - 167 = 527 < 7.5\sqrt{f'_c} = 530 \text{ psi} \therefore OK$

$f_{BOT} = (-M_{DL} - M_{LL} + M_b)/S - P/A$

$= 12000(-694 - 244 + 452)/8416 - 167 = -859 < 0.45 f'_c = 2250 \text{ psi} \therefore OK$

ULTIMATE STRENGTH:

DETERMINE FACTORED MOMENTS — $e = 8 \text{ in}/2 = 8.5/2 = 4.25''$

$M_1 = Pe = \frac{798(4.25)}{12} = 282.6^{'k}$ (PRIMARY PT. MOMENT)

$M_{sec} = M_b - M_1 = 452 - 283 = 169^{'k}$

$M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{sec}$

$M_{u,MIDSPAN} = 1.2(555) + 1.6(195) + 1.0(84.5) = 1062.5^{'k}$

$M_{u,SUPPORT} = 1.2(-694) + 1.6(-244) + 1.0(169) = -1054^{'k}$

DETERMINE MINIMUM BONDED REINF.

POSITIVE MOM. REGION:

INT. SPAN: $f_T = 15 \text{ psi} < 2\sqrt{f'_c} = 2\sqrt{5000} = 141 \text{ psi}$

$\therefore$ NO REINFORCEMENT NEEDED PER ACI 18.9.3.1

NEGATIVE MOM. REGION:

INT. SUPPORTS: $A_{cf} = \max(10\tfrac{1}{2}'')(30)(12) = 3780 \text{ in}^2$

$A_{s,min} = 0.0075(3780) = 2.835 \text{ in}^2$

$\therefore$ 10 #5 TOP (3.1 in^2)

CHECK MIN. REQ'D:

$\phi M_n = \phi(A_s f_y + A_{ps} f_{ps})(d - a/2) \qquad d = 10.5'' - 3/4 - 5/16 = 9.44''$

$A_{ps} = 0.153 \text{ in}^2 (30 \text{ tendons}) = 4.59 \text{ in}^2$

$f_{ps} = f_{se} + 10000 + \frac{f'_c bd}{300 A_{ps}} = 184000 + \frac{(38.167 \times 12)(5000)(9.44)}{300(4.59)}$

$f_{ps} = 200 \text{ ksi}$

$a = \frac{A_s f_y + A_{ps} f_{ps}}{0.85 f'_c b} = \frac{(3.1)(60) + (4.59)(200)}{0.85(5)(38.167 \times 12)} = 0.57''$

$\phi M_n = 0.9\left((3.1)(60) + 4.59(200)\right)\left(9.44 - \frac{0.57}{2}\right) = 9096^{\text{in-k}} = 758^{1k}$

$= 758^{1k} < 1062^{1k}$ ∴ REINF. FOR ULTIMATE STRENGTH CONTROLS

$1062(12) = 0.9\left(A_{s_{req}}(60) + 4.59(200)\right)(9.44 - 0.57/2)$

$\Longrightarrow A_{s_{req}} = 10.5 \text{ in}^2$

∴ USE 24 #6 TOP @ INT SUPPORTS E-W FRAME
$(A_s = 10.56 \text{ in}^2)$

DESIGN OF N-S INTERIOR FRAME

$A = bh = 30 \times 12 \times 10.5'' = 3780 \text{ in}^2$

$S = \frac{bh^2}{6} = \frac{30 \times 12 \times (10.5)^2}{6} = 6615 \text{ in}^3$

$w_b = 0.75\, w_{DL} = \frac{0.75(98)(30)}{1000} = 2.205 \text{ klf}$

$P = \frac{w_b L^2}{8 a_{int}} = \frac{2.205(38.167)^2}{8(8.5/12)} = 567 \text{ kips}$

USE 43 TENDONS

$P_{ACTUAL} = 43(26.6^k/\text{TENDON}) = 1143 \text{ kips}$

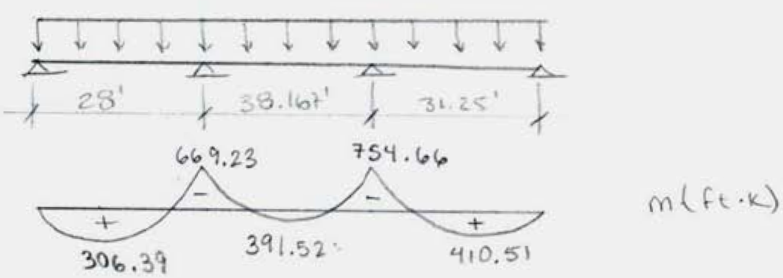
$w_b = \frac{1143}{567}(2.205) = 4.45 \text{ klf}$

$P/A = \frac{1143(1000)}{10.5 \times 30 \times 12} = 302 \text{ psi}$ $\left.\begin{matrix} > 125 \text{ psi min} \\ < 300 \text{ psi max} \end{matrix}\right\}$ WILL ASSUME 302 psi IS OK

EFFECTIVE PRESTRESS FORCE $= 1143^k$ FOR N-S FRAME

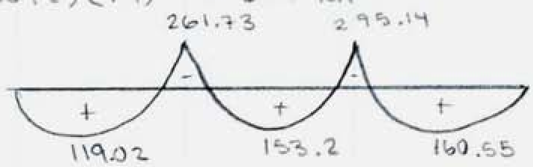
DEAD LOAD MOMENTS:

$w_{DL} = (30 \text{ ft})(131 + 71) = 6.060 \text{ klf}$



LIVE LOAD MOMENTS:

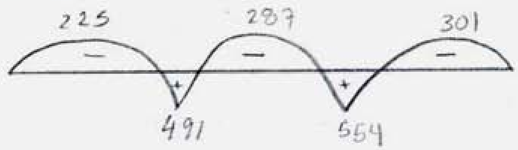
$\omega_{LL} = (30\text{ ft})(79) = 2.370$ klf



m (ft·k)

TOTAL BALANCED MOMENT:

$\omega_b = -4.45$ klf



m (ft·k)

OUTPUT FROM SAP2000 AVAILABLE UPON REQUEST

STAGE 1: STRESSES IMMED. AFTER JACKING (DL+PT)

MIDSPAN

$f_{TOP} = (-M_{DL} + M_b)/S - P/A$

$= 12000(-391.52 + 287)/6615 - 302 = -386 < 1800$ psi ∴ OK

$f_{BOT} = (+M_{DL} - M_b)/S - P/A$

$= 12000(391.52 - 287)/6615 - 302 = -89$ psi < 164 psi ∴ OK

SUPPORT

$f_{TOP} = (M_{DL} - M_b)/S - P/A$

$= 12000(669.23 - 491.0)/6615 - 302 = +16 < 329$ psi ∴ OK

$f_{BOT} = (-M_{DL} + M_b)/S - P/A$

$= 12000(-669.23 + 491.0)/6615 - 302 = -491 < 1800$ psi ∴ OK

STAGE 2: STRESSES @ SERVICE LOAD (DL+LL+PT)

MIDSPAN

$f_{TOP} = (-M_{DL} - M_{LL} + M_b)/S - P/A$

$= 12000(-391.52 - 153.2 + 287)/6615 - 302 = -604$ psi < 2250 ∴ OK

$f_{BOT} = (+M_{DL} + M_{LL} - M_b)/S - P/A$

$= 12000(391.52 + 153.2 - 287)/6615 - 302 = 129 > 424$ ∴ OK

SUPPORT

$f_{TOP} = (+M_{DL} + M_{LL} - M_b)/S - P/A$

$= 12000(669.23 + 261.73 - 491.0)/6615 - 302 = 389 < 530$ ∴ OK

$f_{BOT} = (-M_{DL} - M_{LL} + M_b)/S - P/A$

$= 12000(-669.23 - 261.73 + 491.0)/6615 - 302 = -865 < 2250$ ∴ OK

ULTIMATE STRENGTH

DETERMINE FACTORED MOMENTS

$m_1 = Pe = \frac{1143\,(4.25)}{12} = 405^{\prime k}$

$m_{sec} = m_b - m_1 = 491 - 405 = 86^{\prime k}$

$m_{u,MIDSPAN} = 1.2(306.39) + 1.6(119.02) + 1.0(43) = 601.1^{\prime k}$

$m_{u,SUPPORT} = 1.2(-669.23) + 1.6(-261.73) + 1.0(86) = -1136^{\prime k}$

DETERMINE MINIMUM BONDED REINF.

POSITIVE MOM. REGION

NO REINF. NEEDED

NEGATIVE MOM. REGION

$A_{cf} = 10.5''\left(\frac{38.167 + 31.25}{2}\right) \times 12 = 4373\ in^2$

$A_{s_{min}} = 0.00075\,(4373) = 3.28\ in^2 \therefore$ USE 17 #4 TOP $(A_s = 3.4)$

CHECK MIN REQ'D $d = 10.5 - 3/4 - 1/4 = 9.5''$

$A_{ps} = 0.153\,(43 \text{ TENDONS}) = 6.579\ in^2$

$f_{ps} = 184000 + \frac{5000(30 \times 12)(9.5)}{300\,(6.579)} = 193$ ksi

$a = \frac{(3.40)(60) + (6.579)(193)}{0.85(5)(30 \times 12)} = 0.96$

$\phi M_n = 0.9(3.4(60) + 6.579(193))\left(9.5 - \frac{0.96}{2}\right) = 11964^{\prime\prime k}$
$= 997^{\prime k}$

$= 997^{\prime k} < 1136^{\prime k} \therefore$ REINF FOR ULTIMATE STRENGTH CONTROLS

$1136(12) = 0.9\left(A_{s_{req}}(60) + 6.579(193)\right)\left(9.5 - 0.96/2\right)$

$\Rightarrow A_{s_{req}} = 6.82\ in^2$

$\therefore$ USE 16 #6 TOP @ INT SUPP. N-S FRAME
$(A_s = 7.04\ in^2)$

CHECK PUNCHING SHEAR

ASSUME 18" x 18" COL

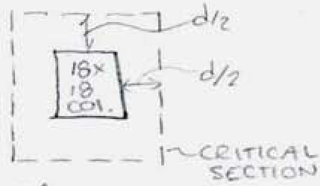
$d = 10.5" - 3/4" - 1/2(0.5") = 9.5"$

$$V_c = \min \text{ of } \begin{cases} 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}\,(110)(9.5)/1000 = 296^k \leftarrow \text{CONTROLS} \\ (2 + 4/\beta_c)\sqrt{f'_c}\, b_o d = (2+4)\sqrt{5000}\,(110)(9.5)/1000 = 443^k \\ \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40}{11.6} + 2\right)\sqrt{5000}\,(110)(9.5) = 403^k \end{cases}$$

$\alpha_s = 40$ for int col

$b_o = (18 + 9.5)4 = 110"$

$b_o/d = 110/9.5 = 11.6"$



DEAD LOAD $= 1.2\left(10.5 \times 150\text{pcf}/12 + 71\right) = 243$ psf

$LLRed = 0.25 + \frac{15}{\sqrt{3970}} = 0.49$

LIVE LOAD = 49 psf

$V_u = 0.243 \text{ ksf}\left(992.5 - \frac{18 \times 18}{144}\right) = 241^k$

$\phi V_n = 0.75(296) = 222^k$

$V_u = 241^k > \phi V_n = 222^k$ ∴ NEED SHEAR REINF.

$\frac{241(1000)}{0.75} = 4\sqrt{5000}\, b_o\,(10.5")$

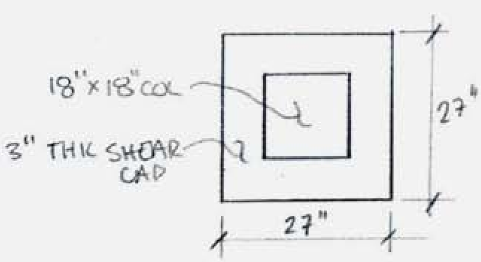
$\Rightarrow b_o = 108"$

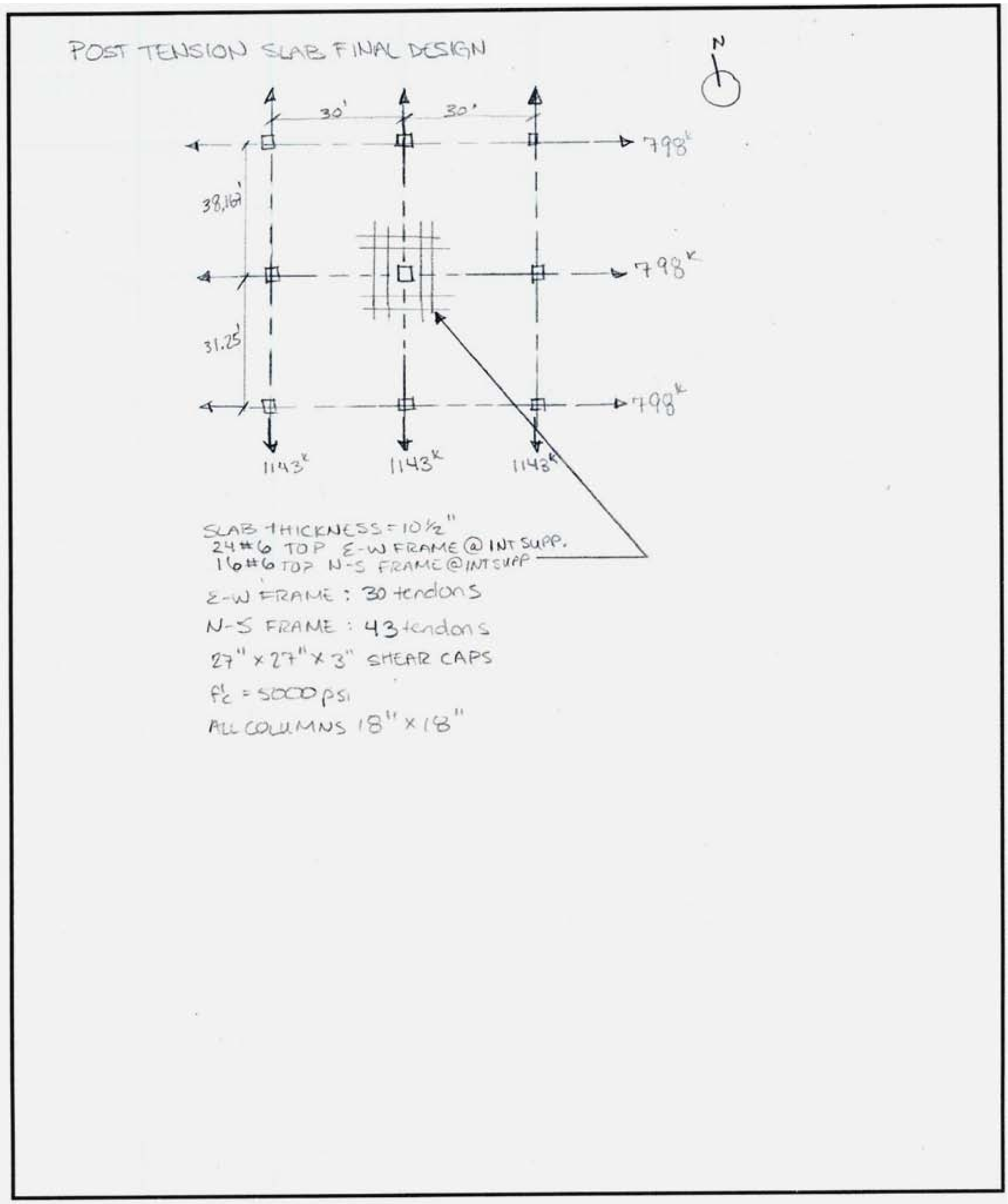
$\frac{108"}{4} = 18 + d \quad \Rightarrow d = 9"$ ∴ shear cap extends 4½" from COL. FACE ON ALL SIDES; USE 27"X27" SHEAR CAP

PER ACI 13.2.5

$t_{cap} \geq 1/4\, t_{slab}$

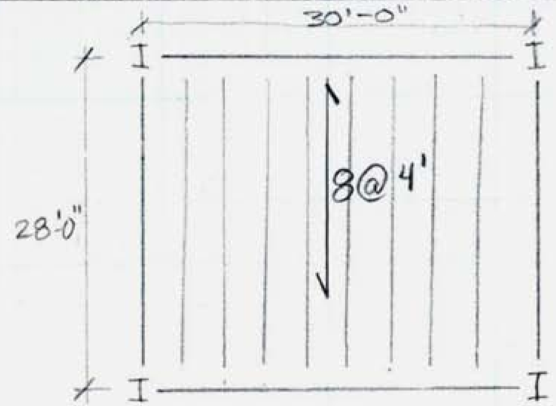
$t_{cap} = 1/4(10.5) = 2.625"$ ∴ USE $t = 3"$




POST TENSION SLAB FINAL DESIGN
N
30'
30'
798k
798k
798k
38.16'
31.25'
1143k
1143k
1143k
SLAB THICKNESS = 10½"
24 #6 TOP E-W FRAME @ INT SUPP.
16 #6 TOP N-S FRAME @ INT SUPP
E-W FRAME : 30 tendons
N-S FRAME : 43 tendons
27" x 27" x 3" SHEAR CAPS
f'c = 5000 psi
ALL COLUMNS 18" x 18"

System 4: Precast Hollow Core Plank on Steel Beams

PRECAST HOLLOW CORE PLANKS ON STEEL BEAM



TYPICAL INTERIOR BAY
30' x 28'
$f'_c = 6000$ psi
$f_y = 50$ ksi

LOADING

SERVICE DEADLOAD = 20psf
LIVELOAD = 100 psf → 79 psf w/ LRed } 99 psf

DESIGN PLANK

$W_u = 1.2(20) + 1.6(79) = 150$ psf

TRY NITTERHOUSE 10" x 4'-0" HOLLOW CORE TOPPED PLANK

- @ 28' SPAN PROVIDES 200 psf
- 2 HR FIRE RATING, 2 in. TOPPING
- $S.W. = 68\text{ psf} + \frac{2}{12}(115\text{pcf}) = 87.17$ psf
- $W_u = 1.2(20 + \frac{2}{12} \cdot 115) + 1.6(79) = 173.4\text{ psf} < 200\text{ psf}$ ∴ OK

USE NITTERHOUSE 10" x 4'-0" w/(7) - ½" ϕ STRANDS
w/ 2" OF LT. WT. CONC.
28' LENGTH

@ 30' : $\Delta_{LL,max} = \frac{\ell}{360} = 1" = \frac{5(79)(28)(30)^4(1728)}{384(29000)\, I_{min}} \Rightarrow I \geq 1390\text{ in}^4$

$\Delta_{DL} = \frac{\ell}{240} = 1.5" = \frac{5(79+87)(28)(30)^4(1728)}{384(29000)\, I_{min}} \Rightarrow I \geq 1947\text{ in}^4$

FOR W24x76 $\phi M_n = 750 > M_u = 546^{k}$

$M_u = \frac{(173.4 \times 28/1000)(30)^2}{8} = 546^{k}$

$I_{W24x76} = 2100 > 1947\text{ in}^4$ ∴ OK

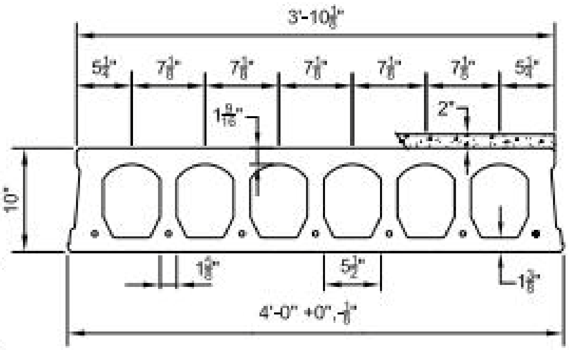
USE W24X76 GIRDERS W/ HOLLOW CORE PLANK

Prestressed Concrete
10"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 327 in.^{2}	Precast b_w = 13.13 in.
I_c = 5102 in.^{4}	Precast S_{bcp} = 824 in.^{3}
Y_{bcp} = 6.19 in.	Topping S_{tct} = 1242 in.^{3}
Y_{tcp} = 3.81 in.	Precast S_{tcp} = 1340 in.^{3}
Y_{tcp} = 5.81 in.	Precast Wt. = 272 PLF
	Precast Wt. = 68.00 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 168.1 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 191.7 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	202	181	161	144	128	114	101	90	79	69	60	52	45	38					
7 - 1/2"ø	LOAD (PSF)	246	222	200	180	162	146	131	118	105	94	84	74	66	58					

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

11/03/08

10F2.0T

Cost Analysis

System 1: Composite and Noncomposite steel beam and deck floor system

Noncomposite Floor System
Using RS Means 2009, Assemblies Cost Data p.94
NYC Location Factor = 1.313

($35.05 /sq ft) x (1.313) = $46.02 / sq ft

$46.02 / square foot

Composite Floor System
Using RS Means 2009, Assemblies Cost Data p.96
($24.70 /sq ft) x (1.313) = $32.43 / sq ft

$32.43 / square foot

System 2: 2way R/C slab with drop panels

Cast in Place Flat Slab with Drop Panels
Using RS Means 2009, Assemblies Cost Data p.66
For 30x 35 bay size
($19.90/sq ft) x (1.313) = $26.13

$26.13 / square foot

System 3: Two-way Post-Tensioned conc. Slab

Using RS Means 2009, Facilities construction Cost Data p.78

Prestressing Steel = $3.33/lb

Cast in Place Concrete = ($575/ CY) x (1CY/27ft^3) x (10.5"/12"/ft)= $18. 63 / sq ft

Tendons

($3.333 /lb)(1244.12 lb) / (30' x 34.71') = $3.98 /sq ft

Strand weight=0.52lb/ft for ½' diam. Strand

0.52lb/ft (30(30) + (43(34.71)) =1244.12 lb

Total Cost = ($18. 63 + $3.98) x (1.313) = $29.69 / square foot

System 4: Precast Hollow Core Plank on steel beam

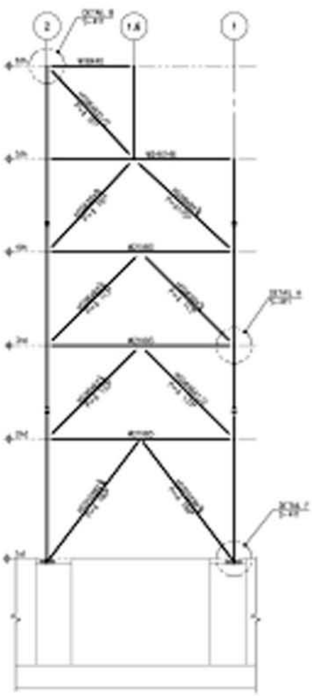
Using RS Means 2009, Assemblies Cost Data p.78
For precast beam and plank with 2" topping
30x30 bay

$25.85	(per square foot (material and installation)
-$11.30	(cost of precast T-beam in assembly)
+$ 13.42	(cost of W shape in 30x30)
$27.97 /sq ft x(1.313)	

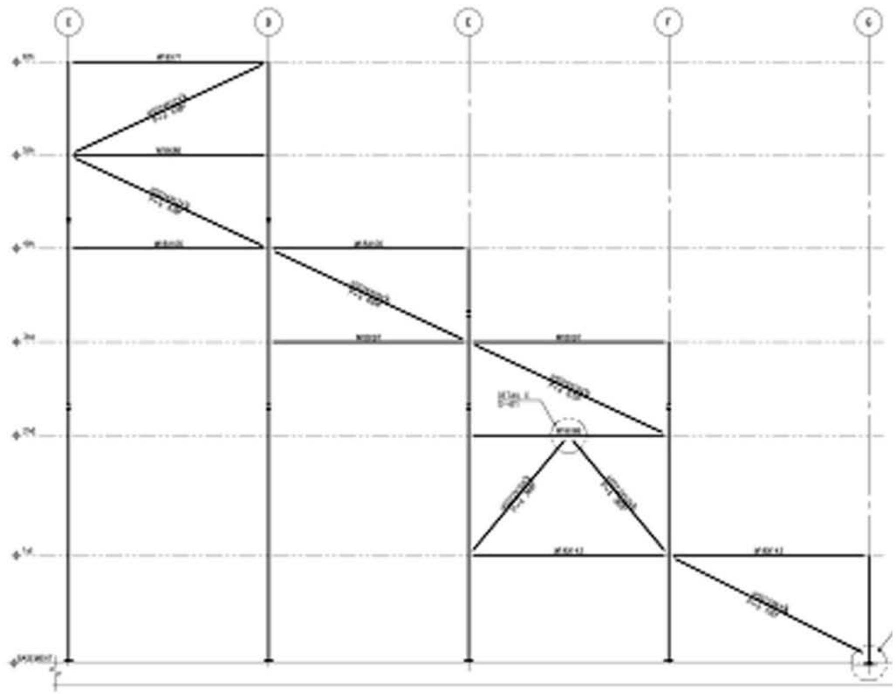
=$ 36.72 /square foot

$0.94	(12x20 precast tbeam)
$6.12	(installation labor an dequipment)
$0.94	(12x20 precast L-Beam)
+$3.30	(installation labor an dequipment)
$11.30	(cost of precast T-beam in assembly)

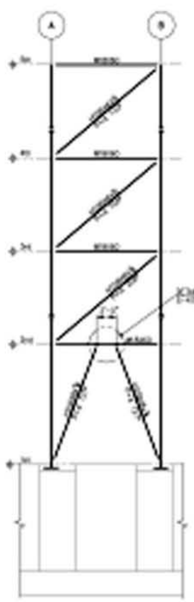
Appendix B - Braced Frames



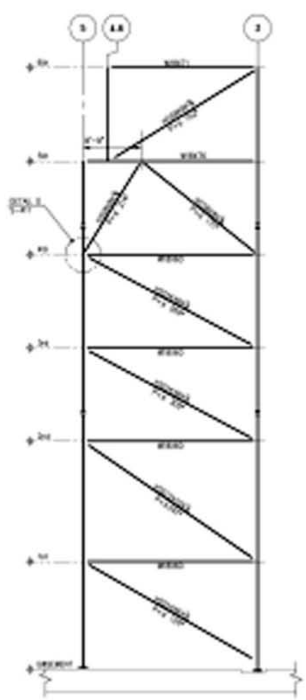
1 TRUSS @ GRID A



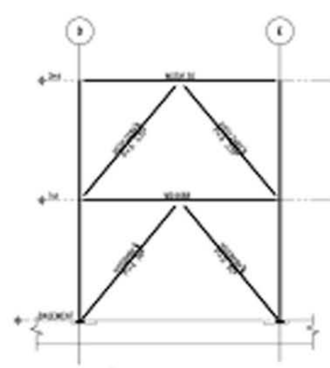
2 TRUSS @ GRID 2



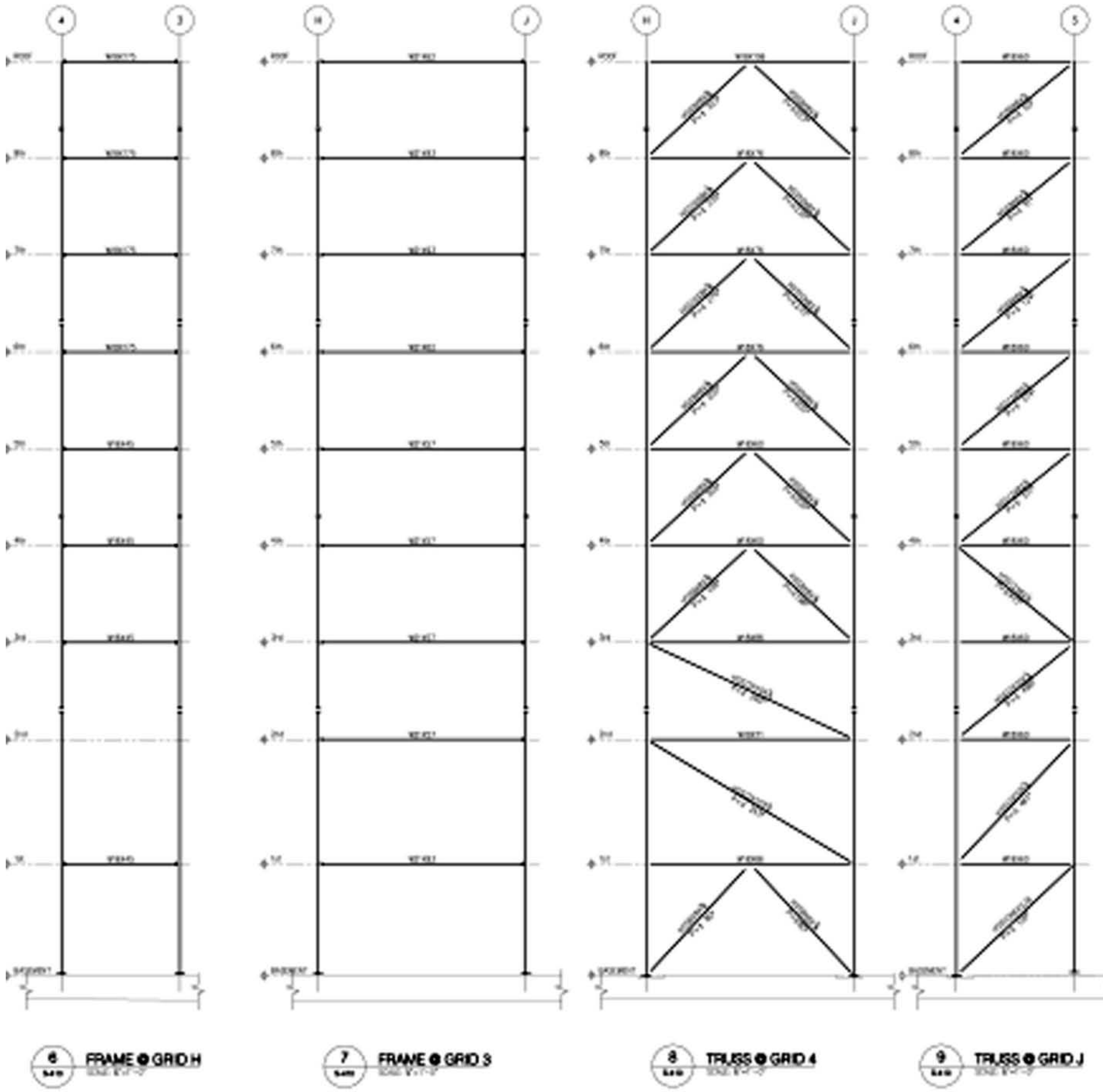
3 TRUSS @ GRID 1



4 TRUSS @ GRID F



5 TRUSS @ GRID B



FRAME @ GRID H
FRAME @ GRID 3
TRUSS @ GRID 4
TRUSS @ GRID J

Appendix C. Loading Diagrams

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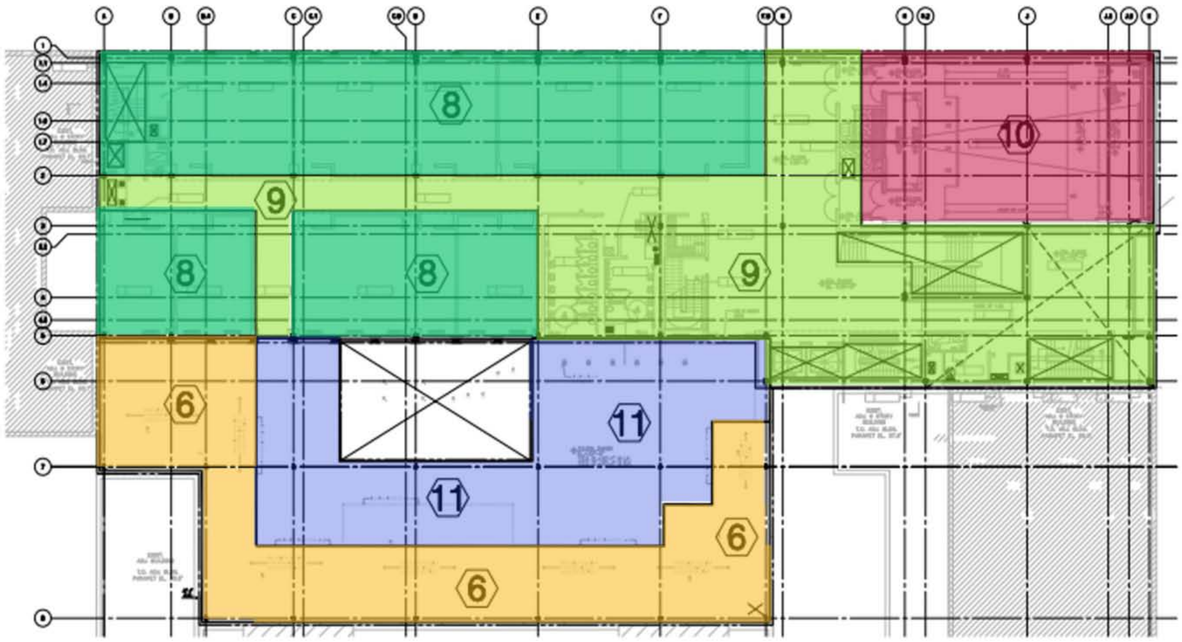
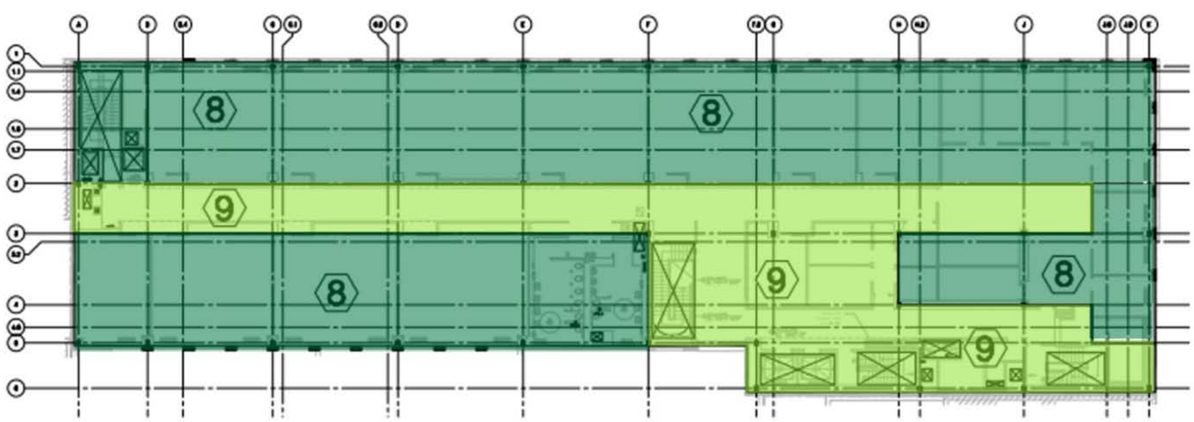
LOADING SCHEDULE		
ID	DL psf	LL psf
1. LOADING DOCK	150.0	600.0
2. 1ST FLOOR	130.0	100.0
3. PODIUM	200.0	100.0
4. ARCHIVE	75.0	350.0
5. OFFICES	71.0	50.0
6. ROOF WITH GARDEN	365.0	100.0
7. LIBRARY STACKS	71.0	100.0
8. CLASSROOMS	71.0	40.0
9. CORRIDOR	71.0	100.0
10. AUDITORIUM	85.0	60.0
11. ROOF WITH PAVERS ON 2	150.0	100.0
12. ROOF	90.0	45.0
13. ROOF WITH DRIFT	85.0	60.0
14. MECHANICAL	120.0	100.0

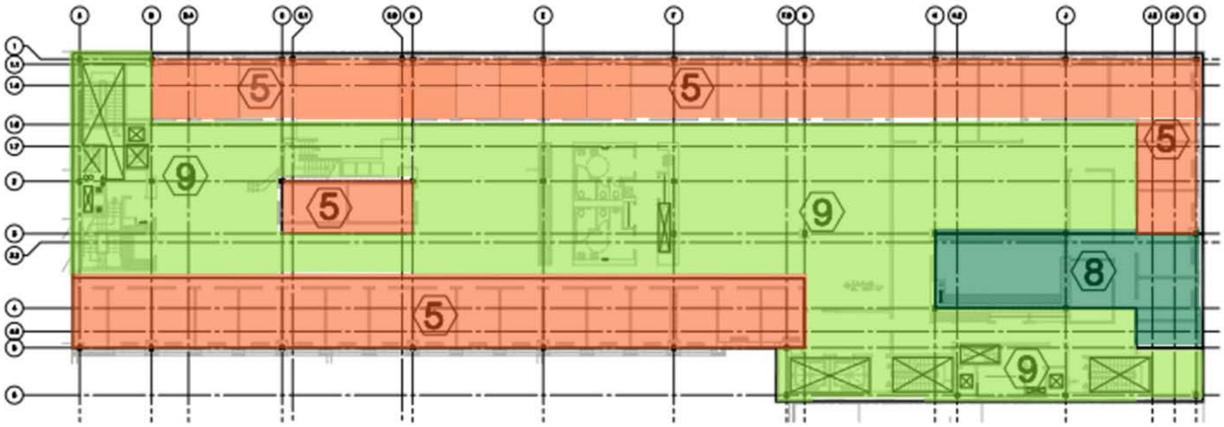
1 1ST FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



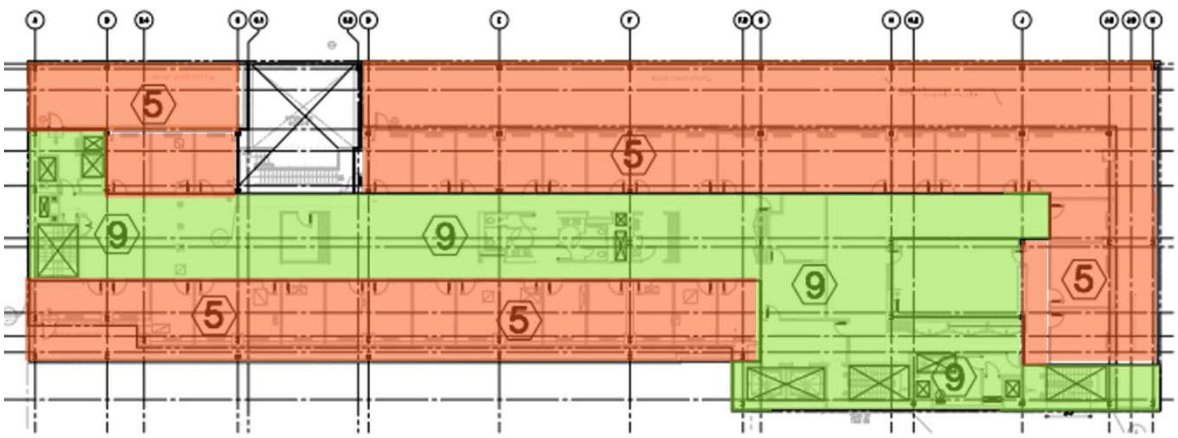
2 2ND FLOOR LOADING DIAGRAM
SCALE: 1/8"=1'-0"



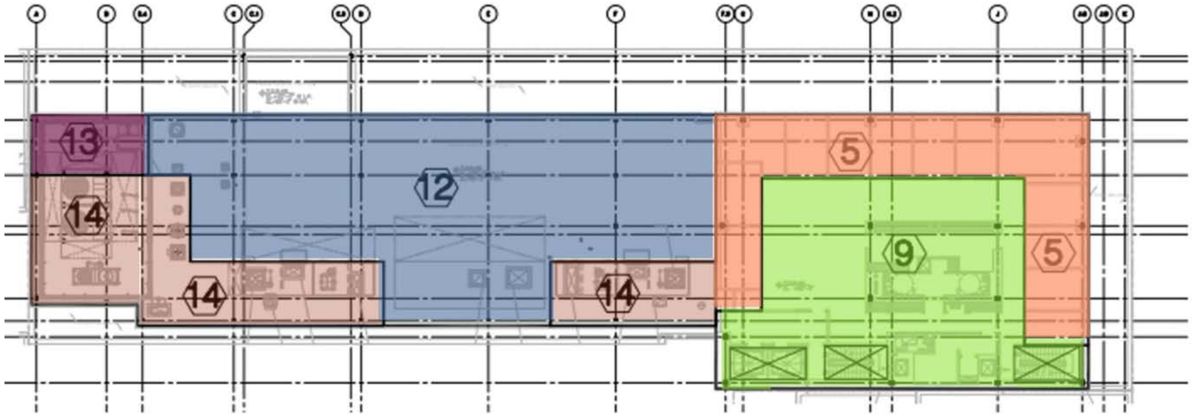
3 3RD FLOOR LOADING DIAGRAM
SCALE: 1/8"=1'-0"



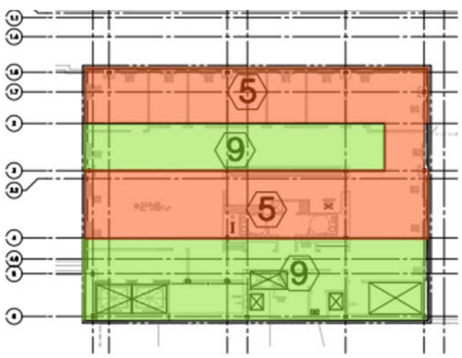
4TH FLOOR LOADING DIAGRAM
SCALE: 1/8"=1'-0"



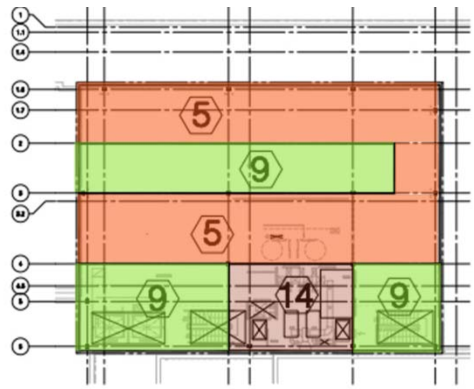
5TH FLOOR LOADING DIAGRAM
SCALE: 1/8"=1'-0"



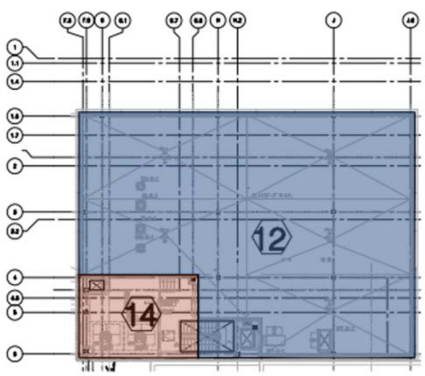
6 6TH FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



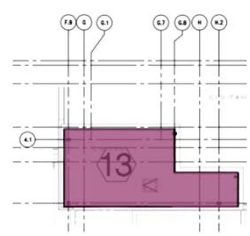
7 7TH FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



8 8TH FLOOR LOADING DIAGRAM
SCALE: ⅛"=1'-0"



9 ROOF LOADING DIAGRAM
SCALE: ⅛"=1'-0"



10 PENTHOUSE LOADING DIAGRAM
SCALE: ⅛"=1'-0"